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GEOTECHNICAL ENGINEERING REPORT

VOLUME I

LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT TASK 2.4.5 –ROUTE 237 TO I-880 SANTA CLARA COUNTY, CALIFORNIA

Prepared for

CH2MHILL
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November 8, 2002
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Mr. David Von Rueden
CH2MHILL
1737 North First Street, Suite 300
San Jose, CA 95112

Subject: Geotechnical Engineering Report
Lower Guadalupe River Flood Control Project
Task 2.4.5 Route 237 to I-880
Santa Clara County, California

Dear Mr. Von Rueden:

In accordance with our Purchase Order/Subcontract 51347 dated August 14, 2001, we are pleased to submit this revised geotechnical engineering report for the Lower Guadalupe River Flood Control Project, to be constructed in Santa Clara County, California. This report addresses the portion of the project alignment between Route 237 and Interstate 880 (I-880), and is supplemental to our previous engineering report entitled "Geotechnical Investigation Lower Guadalupe River Flood Control Project," dated April 28, 2000.

The accompanying report presents our opinions and recommendations regarding the geotechnical aspects influencing design and construction of the proposed levee improvements. The opinions and recommendations have been based upon review of available information, the results of a field investigation, laboratory testing, engineering analyses we conducted as part of our scope, as well as engineering judgment and local experience.

Mr. Thomas Pennington, Staff Engineer, and Ms. Jennifer Werle, Staff Engineer, assisted in the selection of the engineering parameters, the engineering analysis of levee stability and potential levee seepage, as well as preparation of the report. Mr. Pennington also analyzed potential settlement of the levees. Mr. Michael L. Larson, Consulting Geotechnical Engineer, provided peer review for all geotechnical aspects of the project. This peer review was in accordance with the URS Corporation Quality Control Plan.

Since our report of July 19, 2002 was issued, we received review comments from CH2MHILL design team members on September 5, 2002 and met with the team members on September 18, 2002 to discuss the comments. We also received comments from the Santa Clara Valley Water District staff on September 20, 2002 and met with the District staff on October 11, 2002 to discuss their comments. On September 17, 2002, we received a set of plans stamped "Preliminary, Current as of September 17, 2002." Subsequently, we have incorporated all review comments and updated this report to

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include the geotechnical design parameters consistent with the latest project data we have received.

We are pleased to have been of service to you on this project. If any questions should arise, please contact our office.

Sincerely,

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Deputy Project Manager

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VOLUME II

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This report presents the results of a geotechnical investigation for the Lower Guadalupe River Flood Control project for Levee Improvements from Route 237 to Interstate 880 (I-880). The location and layout of the project are shown in Figure 1, Site Map. A more detailed layout of the project, including the approximate centerline of the existing West Levee and East Levee is shown in Figure 2, Site and Field Exploration Location Plan. This figure includes the design control line with stationing presented in meters, as well as estimated limits of historical meanders, and locations of existing and new field exploration points. Locations of streets and highways crossing the Guadalupe River are also shown in Figure 2. Topographic maps with aerial photography background were presented as part of our April 28, 2000 report for this project. Throughout the text of this report, elevation, distance and stationing will be presented in metric units; except for stationing, English units will also be presented in parenthesis. According to the latest design drawings with sheets dated June 28, July 1 and July 3, 2002 (referred as July 2002 Design Drawings from this point onwards) there are three separate station number systems, including the "P" Line, "E" Line and "W" Line. The "P" Line traverses the approximate centerline of the channel; the station numbers presented in this report are based on the "P" Line unless stated otherwise. The "E" Line traverses the crest of the East Levee, i.e. looking upstream. The "W" Line traverses the crest of the West Levee, i.e. looking upstream. Most graphs, charts and laboratory test results are presented in English units.

The report presents the results of (1) a review of the available subsurface information, construction records and as-built drawings, (2) the subsurface exploration and laboratory testing program conducted for this study, and (3) the results of engineering evaluations that serve as the basis for our opinions and recommendations regarding the geotechnical engineering aspects of the project.

The goal of this study is to provide a design level geotechnical investigation for the Lower Guadalupe River in order to bring the levees into compliance with the requirements of the Federal Emergency Management Agency (FEMA) and the United States Army Corps of Engineers (Corps). Ultimately, the Santa Clara Valley Water District (District) wishes to obtain FEMA certification of the levees in order to receive a Letter of Map Revision (LOMR).

To meet FEMA standards, the levees must provide a minimum freeboard of 0.9 meter (3 feet) above the 100-year (1-percent) flood. An additional 0.3 meter (1 foot) above the minimum is required within 30.5 meters (100 feet) on either side of structures (such as bridges) riverward of the levees or wherever the flow is constricted. An additional 152 mm (1/2 foot) above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. The District also must demonstrate that the levees are stable from a geotechnical standpoint.

1.1 PROJECT DESCRIPTION

The Lower Guadalupe River has been defined as that portion of the river that extends from I-880 to the river's terminus with the San Francisco Bay at Alviso. This portion of the Guadalupe River is located in the City of San Jose and Santa Clara County,

California. This report addresses the portion of the levee alignment between Route 237 (Station 8+000) and I-880 (Station 17+500). Consequently, the levees analyzed for this study extend approximately 9,500 meters (31,200 feet). The District currently proposes to meet the freeboard requirements with the construction of floodwalls up to 1.65 meters (5.41 feet) high above existing grade, and levee raising between Stations 8+700 and 17+500. More specifically, the levee will be raised a maximum of 1.9 meters (6.2 feet) between Route 237 (Station 8+700) and the Airport Island Bridge (about Station 14+700); in general the levee crest raise decreases in magnitude in a southerly (upstream) direction. Although the maximum rise is 1.9 meters (6.2 feet), the maximum rise is typically less than 1.1 meters (3.6 feet). Between the Airport Island Bridge (Station 14+700) and I-800 (Station 17+612), the levees will not be raised, except as follows:

- West Levee, near Station 15+500, about 240 meters (787 feet) long, maximum rise height of 0.6 meter (2.0 feet).
- West Levee, near Station 16+000, about 230 meters (754 feet) long, maximum rise height of 1.2 meters (3.9 feet).
- West Levee, near Station 16+815, about 10 meters (32.8 feet) long, maximum rise is 0.9 meter (3.0 feet).
- West Levee, near Station 17+100, about 160 meters (525 feet) long, maximum rise is 0.6 meter (2.0 feet).

Where existing levees are to be raised, most of the finished side slopes are inclined at 2:1 (horizontal to vertical) on both the landside and riverside. In addition, a vortex rock weir is planned in the channel just downstream of I-880 between approximately Stations 17+420 to 17+505. This weir consists of four walls of partially protruding 1.22 meters (4 foot) diameter boulders.

A summary of bridge locations and geotechnical analysis comments is presented in Table 1. The comments and assumptions presented are based on the October 30, 2001 workshop.

According to Sheets S-1 through S-24, CD-42, CD-43, CD-64 and Cd-65 of the July 2002 Design Drawings, floodwalls are planned with the lengths and heights as described in the following table.

Floodwall	Station	Location	Length (m)	Wall Height (mm)		Sheet Numbers	Maximum Fill Height (m)
				Minimum	Maximum		
FW5E	E22+292.000 To E22+729.883	East levee, south of Route 237 bridge	438.235	1,200	2,100	S-10 to S-13	0.7
FW4W	W41+449.000 To W41+842.576	West levee, south of Route 237 bridge	393.521	1,200	2,400	S-20 to S-22	0.8

Floodwall	Station	Location	Length (m)	Wall Height (mm)		Sheet Numbers	Maximum Fill Height (m)
				Minimum	Maximum		
FW6W	W43+341.192 To W43+430.531	West levee, north of Hetch-Hetchy Pipeline Crossing	79.339	1,200	1,200	S-24	0.5
FW7E	E25+826.724 To E25+855.304	East levee, north of Montague Expressway bridge	29.369	1,200	1,800	CD-42	1.0
FW8E	E25+888.222 To E25+923.134	East levee, south of Montague Expressway bridge	35.129	1,500	2,100	CD-42	0.8
FW9W	W45+037.670 To W45+061.022	West levee, north of Montague Expressway bridge	23.392	1,200	1,800	Cd-43	1.1
FW10W	W45+093.807 To W45+216.944	West levee, south of Montague Expressway bridge	124.492	1,500	2,100	CD-43	0.6
FW9E	E28+576.110 To E28+580.329	East levee, south of Highway 101 bridge	6.375	1,800	1,800	CD-64	1.8 at bridge abutment; 0.3 remaining
FW10E	E28+634.796 To E28+656.664	East levee, south of Highway 101 bridge	24.450	1,200	1,800	CD-64	0
FW11W	W47+729.763 To W47+732.143	West levee, south of Highway 101 bridge	2.428	1,800	1,800	CD-65	0.4 at bridge abutment; 0 remaining
FW12W	W47+785.219 To W47+813.617	West levee, southwest of Highway 101 bridge	28.767	1,500	1,800	Cd-65	0

Based on the above table, the total length of floodwalls is about 1,185 meters (3,890 feet). Wall height H ranges from 1.2 to 2.4 meters (3.9 to 7.9 feet); H is defined from top of floodwall to top of footing. Since the minimum burial depth is 0.75 meter (2.46 feet), the wall height above finished grade ranges from 0.45 to 1.65 meters (1.48 to 5.41 feet).

Furthermore, differential fill height planned behind these floodwalls is nil to 1.3 meters (4.3 feet).

According to the July 20002 Design Drawings, retaining walls are planned with the lengths and heights described in the following table.

Retaining Wall	Station	Location	Length (m)	Height (mm)		Elevation of Bottom of Footing (m)	Sheet Number
				Minimum	Maximum		
RW3E	E23+141.013 to E23+241.001	East levee, north of Tasman Drive bridge	101.000	1,200	3,600	1.700 to 6.400	CD-21
RW4E	E23+288.509 to E23+391.570	East levee, south of Tasman Drive bridge	104.650	1,200	4,200	0.600 to 6.150	CD-22
RW6E	E25+733.010 to E26.024.286	East levee, wall at Montague Expressway	288.500	1,200	4,200	1.800 to 7.7000	CD-34 to CD-35
RW3W	W45+057.115 to Begin Tieback Wall	West levee, north wall at Montague Expressway	3.268	2,400	2,400	2.85	CD-36
RW3W	End Tieback Wall to W45+192.500	West levee, south of Montague Expressway bridge	95.210	1,200	2,400	3.000 to 6.800	CD-36 to CD-37

Based on the above table, the total length of retaining walls is 877.860 meters (2,879 feet). Wall height H ranges from 1.2 to 4.2 meters (3.94 to 13.78 feet). Most of the walls will have a relatively level backfill surface.

Based on the July 2002 Design Drawings, we understand seven mechanically stabilized embankments (MSE) walls are planned with the lengths and heights described in the following table.

MSE Wall	Station	Length (m)	Height Range (H, m)	Strip Length = 1.2*H (m)	Sloping Backfill?
MSW1W	W43+410.000 to W43+520.662	113.000	2.4 – 3.0	2.9 – 3.6	Yes
MSW2E	E24+600.497 to E24+696.814	96.500	2.4 – 3.0	2.9 – 3.6	Yes
MSW2W	W44+560.000 to W44+631.882	74.000	1.8 – 3.0	2.2 – 3.6	Yes
MSW3E	E26+034.420 to E26+859.451	825.000	2.4 – 3.0	2.9 – 3.3	Yes
MSW3W	W46+180.000 to W46+260.010	79.000	1.8 – 3.0	2.2 – 3.6	Yes
MSW4E	E27+295.000 to E27+650.000	356.858	2.4 – 3.0	2.9 – 3.6	Yes
MSW5E	E27+820.000 to E27+918.982	96.500	1.8 – 2.4	2.2 – 2.9	Yes

*Preliminary strap length provided by CH2MHILL.

Based on the above table, the total length of retaining walls is 2,120 meters (6,957 feet). Wall height H ranges from 1.2 to 4.2 meters (3.9 to 13.8 feet). Most of the walls will have a sloping backfill surface.

At the Tasman Bridge, a maintenance road is planned underneath the bridge. Steepening of the slope at each abutment by excavation into the West and East Levees will be required. The new side slope will be 2:1 and will be covered with a concrete pavement.

For similar reasons, tieback walls will be installed at four locations under three existing bridges, including Montague Expressway West Levee, Highway 101 East Levee, Highway 101 West Levee and Airport Parkway West Levee. Planned wall heights range from 3.0 to 4.0 meters (9.9 to 13.1 feet).

At Route 237, the eastbound bridge will be removed and replaced with a new bridge to provide adequate clearance between the bridge soffit and 100-year flood water surface. Furthermore, at the existing westbound bridge a new retaining/tieback wall/shear wall is planned on the East Levee/embankment. Geotechnical investigations are presently underway for the new eastbound bridge (Task 2.4.1) and modifications to the westbound bridge (Task 2.4.2); two separate geotechnical reports will be prepared by URS, one for each bridge.

A setback levee is planned on the west levee beginning at approximately Station W47+300 and extending to W47+700; it is approximately 30 meters (98 feet) downstream of Highway 101. Both ends of this setback levee conform to the existing West Levee. The new setback levee will have side slopes with inclination of 2:1 on both the inboard and outboard sides. On the outboard side, the levee crest will be about 1.5 to 1.8 meters (4.92 to 5.90 feet) above the adjacent existing ground surface. On the inboard side, the finished slope will extend to the existing channel bottom by excavating and removing the existing levee section and subgrade soils to create a widened channel; the finished inboard slope will be approximately 5 to 6 meters (16.40 to 19.68 feet) high. The results of the geotechnical investigation are discussed in Section 8 "Highway 101 Setback Levee".

According to District As-built drawings (1995), portions of the existing West and East Levees were raised above their existing crest by (1) placement of "impervious sandy clay, PI less than 25±5", or (2) construction of L-shaped reinforced concrete floodwalls, or (3) placement of engineered fill within layers of geotextile soil reinforcing (GSR). This resulted in steepening of the slopes of the raised portions generally to 1.75 to 1 (horizontal to vertical) on the riverside; whereas slopes on the landside were generally 1.5:1 for clay fill and 1:1 or 1.75:1 for GSR. For GSR levee improvements, 2 or 3 horizontal fabric layers were installed and wrapped around on the outside landside face. The 1995 drawings indicate that "semi-impervious, or structural backfill, PI less than 15±5" was placed inside the outermost 1.5 m to 2.7 m (5 to 9 feet) landside portion of the GSR. The following table describes the locations where GSR was used between Gold Street and I-880.

Levee	Limits of Geotextile Soil Reinforcing (GSR)			Length		Wall Height		Comments
	Begin (feet)	End (feet)	Location	(feet)	(m)	(feet)	(m)	
East	1098+44	1111+30	South of Tasman Drive	1,286	392.0	2.5-3.83	0.76-1.17	5 to 9 feet of "semi-impervious" or "structural backfill"
East	1092+30	1096+27	North of Tasman Drive	397	121.0	2.5-3.83	0.76-1.17	5 to 9 feet of "semi-impervious" or "structural backfill"
West	84+90	97+94	North of Tasman Drive	1,304	397.6	2.5-3.83	0.76-1.17	5 to 9 feet of "semi-impervious" or "structural backfill"
East	1052+00	1058+25	North of Route 237	625	190.6	2±	0.61±	Assumed 5 feet of "semi-impervious" or "structural backfill"
West	45+70	52+50	Between Gold Street and Route 237	680	207.3	2±	0.61±	Assumed 5 feet of "semi-impervious" or "structural backfill"

More specifically, the total length of GSR wall for Task 2.4.4 is 397.9 meters (1,305 feet) and for Task 2.4.5 is 910.6 meters (2,988 feet).

CH2MHILL has designated seven (7) reaches for the entire Lower Guadalupe River Flood Protection Project as described in the following table.

Reach	Station Limits		Description	
	From	To	From	To
G	7+000	8+000	Downstream of UPRR bridge	Downstream of Route 237
F	8+000	9+200	Downstream side of Route 237	Downstream of Tasman Drive
E	9+200	10+100	Downstream of Tasman Drive	Downstream of Hetch Hetchy Aqueduct
D	10+100	11+800	Downstream of Hetch Hetchy Aqueduct	Downstream of Montague Expressway
C	11+800	13+650	Downstream of Montague Expressway	Downstream of Trimble Road
B	13+650	14+500	Downstream of Trimble Road	Downstream of Highway 101
A	14+500	15+510	Downstream of Highway 101	Station 15+510

As this report is for Task 2.4.5 only, the geotechnical engineering analysis, conclusions and recommendations are limited to Reach A through Reach F. Our geotechnical

analysis, conclusions and recommendations for Reach G (Task 2.4.4) were provided in a previous separate report dated July 5, 2002.

As in the previous 2000 study (URS, April 2000), we have used the NGVD survey datum for the elevation of the ground surface at all explorations and subsurface profiles; both NGVD English and Metric units are provided in this report text. In the event that conversion to NAVD is desired in metric units, the following equation should be used:

$$\text{NAVD (metric)} = \text{NGVD (metric)} + 0.81$$

The elevations shown in the August and December 2001 drawings are based on the NAVD datum.

The 100-year (1-percent) flood hydrograph provided by the District and developed by the Corps is shown in Figure 3. As shown, the peak discharge is 481.4 cubic meters per second or cms (17,000 cubic feet per second or cfs); the duration of flow exceeding 453 cms (16,000 cfs) only lasts about 12 hours. When the discharge rate is reduced to 113 cms (4,000 cfs) or about one-quarter of the peak discharge, the time interval is about 42 hours.

1.2 SCOPE OF WORK

The following scope of work has been performed to serve as a basis for the development of the opinions and recommendations presented in this draft report:

- Reviewed available information including current levee cross sections, topographic drawings and design drawings provided by the District and CH2MHILL.
- Reviewed proposed improvements including raised levee, channel toe excavation, floodwalls, tieback walls, and retaining walls/gabions under bridge abutments.
- Selected location of new borings.
- Cleared utilities at field exploration points through Underground Service Alert.
- Performed field exploration including:
  - 4 borings in East Levee (Borings EB-23 through EB-26) to depths of 13.6 m (44½ feet) for Task 2.4.5. In addition, Borings EB-21 and EB-22 were drilled in conjunction with exploration for the Route 237 bridge (Task 2.4.1 and Task 2.4.2) to a depth of 40.1 m (131½ feet).
  - 5 borings in West Levee (Borings WB-21 through WB-25) to depths of 13.4 to 13.6 m (44 to 44½ feet)
  - 9 borings in the channel (Borings CB-05 through CB-13) to depths of 10.2 to 12.7 m (33½ to 41½ feet). In addition, Borings CB-02, CB-03 and CB-04 were drilled in conjuction with exploration for the Route 237 bridge (Task 2.4.1 and Task 2.4.2) to a depth of 36.5 to 37.7 m (120 to 123½ feet).
  - total of 18 borings on levees and in channel for Task 2.4.5; all boreholes were grouted upon completion. Logs for all 22 borings drilled in the vicinity of the task limits are presented in Appendix A for information purposes.
- Performed laboratory testing, including:

  - Grain size distribution
  - Plasticity index
  - Unconfined compression, water content and dry density
  - Unconsolidated undrained triaxial compression
  - Consolidated undrained triaxial compression with pore pressure measurements
  - Consolidation
- Performed preliminary geotechnical assessment.
- Identified key geotechnical cross-sections along both levees.
- Characterized soil profiles for these cross sections.
- Identified potential geotechnical issues.
- Selected seventeen (17) most critical levee cross sections.
- Performed geotechnical calculations:
  - At 17 cross sections, 11 East Levee and 6 West Levee
  - At 14 cross sections with 1.5:1 outboard slope
  - Factors of safety for slope stability for 4 loading conditions
  - Slope stability earthquake deformations
  - For concrete paved slopes at Tasman Bridge
  - Floodwall design parameters
  - Settlement
  - Seepage
  - Tieback wall design parameters
  - Retaining wall and MSE wall design parameters (also included global stability and seepage evaluation of the MSE walls)
  - In accordance with Corps Engineer Manual No. 1110-2-1913 (Design and Construction of Levees) and Engineer Manual No. 1110-2-1904 (Soil Mechanics Design - Settlement Analysis)
- Performed geotechnical investigation for Setback Levee downstream of Highway 101 on west bank
- Prepared draft geotechnical report:
  - Summarized the results of all the above tasks
  - Developed geotechnical opinions and recommendations regarding the structural integrity of both levees.
  - Developed geotechnical design parameters for the reinforced concrete floodwalls
  - Developed geotechnical design parameters for the tieback walls

1.3 PREVIOUS INVESTIGATIONS

We have previously performed an investigation for the subject project. The results of that investigation were presented in a report entitled "Geotechnical Investigation, Lower Guadalupe River Flood Control Project, Santa Clara County, California," dated April 28, 2000. That report included the entire alignment of the project from the Union Pacific Railroad (UPRR) to Interstate 880. We performed slope stability, seepage, and settlement analyses, and provided recommendations based on the selection of seven

critical cross sections. Six cross sections (Cross Section 2 through Cross Section 7) from the April 2000 study are within the limits of this study.

The logs of 185 previous borings completed along the levees or for projects in the immediate vicinity of the levees are provided in Appendix A of the April 2000 report. These logs represent a total of 26 different projects, over the period of 1960 through 1997. The index list for each project is shown in Table 2; a different boring number index is assigned for each project. The locations of each of these borings and CPTs (along with the new borings) are shown in Figure 2, Site and Field Exploration Location Plan. The depth of these borings ranges from 2.4 to 30.8 meters (8 to 101 feet). Most of the existing boring logs are from investigations previously performed by URS' predecessor firm, Woodward-Clyde Consultants. The existing borings shown on Figure 2 were selected based on these two criteria:

- They were located within or in close proximity to the footprint of the levees; and
- The technical information provided on the log was reliable, in our engineering judgment.

It should be noted that borings in the initial investigations in the early 1960s were drilled prior to construction of the levees and therefore reveal subsurface conditions only in the foundation. Subsequent borings drilled mostly in the 1980s and 1990s extended through the levee section and therefore revealed the levee fill and underlying foundation conditions at that time.

Most of the routine laboratory test data are shown on the individual boring logs, i.e. water content, dry density, and unconfined compression. The results of more complex laboratory tests such as grain size distribution, plasticity index, consolidation and consolidated undrained triaxial compression (with pore pressure measurements) from the previous investigations are presented in Appendix B of the April 2000 report.

1.4 DATA REVIEW

As part of the April 2000 investigation, we reviewed the available levee design and construction information, as well as related records (mostly provided by the District) including the following:

- Drawings for the original 1963 river relocation
  - 16 sheets (some as-built) dated 1959 and 1960 and entitled "Map and General Construction Plans of Zone C-1 (Central) Project No. 3, Guadalupe River Improvement, San Francisco Bay to Branham Lane, Santa Clara County Flood Control and Water Conservation District, Santa Clara County California"; most drawings prepared by Creegan and D'Angelo.
  - 54 sheets (some as-built) dated April 26, 1963 and entitled "Map and General Construction Plans of Guadalupe River Unit 1 From Montague Road to Alviso Slough, Zone C-1 (Central), Project No. 30016, Santa Clara County Flood Control and Water Conservation District, Santa Clara County, California"; most drawings prepared by Creegan and D'Angelo.

- As-built drawings for the 1983 levee raising including 116 sheets dated September 27, 1984(as-built) and entitled "Map and Construction Plan for Guadalupe River From S.P.R.R. to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District"; logs of 10 soil borings were included on 3 plan sheets.
- As-built drawings for the 1995 interim raising between UPRR and Highway 101, including 15 sheets dated April 11, 1996 (as-built) entitled "Map and General Plan for Guadalupe River From Southern Pacific Railroad to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District".
- Approximately 240 current levee cross sections.
- Current topographic and aerial photography drawings including 15 sheets dated November 25, 1997; no title; from UPRR. bridge to south of Highway 101 (Airport Island Bridge). Contours are plotted on both levees using a 1.0 meter contour interval.
- Existing elevations of crest of both East and West Levees. Settlement was estimated by subtracting as-built levee crest elevations from the existing elevations.
- A graphical plot of land subsidence from a United States Geological Survey paper entitled "Land Subsidence in the Santa Clara Valley, Alameda, San Mateo and Santa Clara Counties, California", 1971, by J.F. Poland.

We have also reviewed available District drawings and other historical records for locations of meanders. As shown on Figure 2, Site and Field Exploration Location Plan, the approximate channel limits of old meanders are plotted for almost the entire length of the project; these are based on District drawings dated 1960 and 1963. Between Tasman Drive and Montague Expressway, the locations of earlier meanders are based on a 1873 map obtained from the archives of the San Jose Historical Museum ("Map of a Re=Survey of the "Lick" Property, Party, Part of "Rancho Rincon De Las Esteros", surveyed October 31, 1873 by J.H. Pieper and Re=Surveyed July 1887 by H.A. Braissard) were identified and plotted on Figure 2.

During construction of the levees, we understand the District typically required the removal of soft deposits within the meander and within the footprint of the levee and replacement with soil spread in layers and compacted by track walking with a bulldozer or other track-mounted equipment. However, no specific relative compaction requirements were set.

As part of the current investigation, we reviewed the available levee improvement design information, mostly provided by CH2MHILL, including the following:

- "Santa Clara Valley Water District, Lower Guadalupe River Planning Study, Engineer's Report, Project No. 3015" by CH2MHILL, Inc. dated August 2001.
- Drawings for the proposed levee improvements in the plans entitled "Map and general Plan for the Lower Guadalupe River Flood Control Project from Interstate 880 to Union Pacific Railroad (UPRR) Bridge" dated August 2001.
- "Supplemental Design Memorandum, 30-Percent Design Submittal, Lower Guadalupe River Flood Protection Project, Alviso Marina to Interstate 880, Project No. 3301511" by CH2MHILL, dated December 7, 2001.

- Drawings for the proposed levee improvements in the plans entitled "Map and Construction Plan for Lower Guadalupe River Flood Protection Project from the Alviso Marina to Interstate 880", Project No. 301511, dated December 7, 2001.
- Drawings for the proposed levee improvements in the plans entitled "Map and Construction Plan for the Lower Guadalupe River Flood Protection Project from the Alviso Marina to Interstate 880", Project No. 301511, dated April 29, 2002.

This section was included in the April 2000 report, and is repeated herein for ease of reference. No additional geologic study was performed as part of the current investigation, except Section 2.3 "Ground Motions" have been modified.

2.1 GEOLOGIC SETTING

The levees span from the northerly end of the Santa Clara Valley in Alviso to nearly the heart of the Valley in San Jose; the project area lies between the Santa Cruz Mountains to the west and the Diablo Range to the east. The Santa Clara Valley is a broad northwesterly trending alluvial filled basin. The older Mesozoic rocks of the Santa Cruz Mountains and the Diablo range underlie the thick accumulations of alluvial sediments in the Santa Clara Valley. These alluvial sediments are divided into the lower older Santa Clara Formation, and the upper younger surficial deposits of alluvium, and alluvial fans.

Depth to bedrock is estimated to be at least 92 meters or 300 feet (Rogers and Williams, 1974) along the levee alignment.

2.2 REGIONAL FAULTS AND SEISMIC SOURCES

The active faults that could potentially generate earthquakes producing strong ground motions along the Guadalupe River Flood Control Improvement Project are described below and are shown on Figure 4, Regional Active Faults. Each fault is characterized in terms of the type of faulting, rupture length, dip, maximum magnitude, and slip rate. The 84th percentile peak horizontal accelerations potentially generated by the maximum earthquake for each seismic source are tabulated in Tables 3a to 3d. The maximum magnitudes for these faults were adopted from the Working Group on Northern California Earthquake Potential (WGNCEP, 1996).

2.2.1 San Andreas Fault

The San Andreas fault extends from the Gulf of California, Mexico, to Point Delgada on the Mendocino Coast in northern California, a total distance of 1,200 km (746 miles). The San Andreas accommodates the majority of the motion between the Pacific and North American plates. This is the largest active fault in California and is responsible for the largest known earthquake in northern California, the 1906 moment magnitude (M_w) 8 San Francisco earthquake (Wallace, 1990). Movement on the San Andreas fault is right-lateral strike-slip, with a total offset of some 560 km or 348 miles (Irwin, 1990). In northern California, the San Andreas fault is clearly delineated, striking northwest, approximately parallel to the vector of plate motion between the Pacific and North American plates. Over most of its length, the San Andreas is a relatively simple, linear fault trace. However, immediately south of San Francisco Bay, the fault splits into a number of fault splays, including the Calaveras and Hayward faults. In the San Francisco Bay area the main trace of the San Andreas fault forms a linear depression along the peninsula occupied by the Crystal Springs reservoirs. Geomorphic evidence for Holocene faulting includes fault scarps in Holocene deposits, right-laterally offset streams, shutter ridges and closed linear depressions (Wallace, 1990). The 1906 earthquake resulted from rupture of the fault from San Juan Bautista north to Point

Delgada, a distance of approximately 470 km (293 miles). The average amount of slip on the fault during this earthquake was 5.1 meters (16.7 feet) in the area to the north of the Golden Gate, and 2.5 meters (8.2 feet) in the Santa Cruz Mountains (WGNCEP, 1996).

Based on differences in geomorphic expression, fault geometry, paleoseismic chronology, slip rate, seismicity, and historic fault ruptures, the San Andreas fault is divided into a number of fault segments. In the San Francisco Bay Area, these segments include the Santa Cruz Mountains, possible source of the 1989 M_w 7.1 Loma Prieta earthquake and the Peninsula segment. The slip rate on these segments is 14 and 17 mm/year (0.55 and 0.67 inch/year) respectively. Both of these segments, along with the North Coast segment, ruptured during the 1906 M_w 8 San Francisco earthquake. The Santa Cruz Mountains, Peninsula, and 1906 rupture segments have calculated maximum credible earthquakes (MCEs) of M_w 7, 7, and 8, respectively (WGNCEP, 1996).

2.2.2 Hayward Fault

The Hayward fault extends for 100 km (62 miles) from the area of Mount Misery, east of San Jose, to Point Pinole on San Pablo Bay. Systematic right-lateral geomorphic offsets and creep offset of cultural features have been well documented along the entire length of the fault (e.g., Lienkaemper et al., 1991). The Hayward fault is considered to be the most likely source of the next major earthquake in the San Francisco Bay area (WGCEP, 1990). The last major earthquake on the Hayward fault, in October 1868, occurred along the southern segment of the fault. This Richter local magnitude (M_L) 7 event caused toppling of buildings in Hayward and other localities within about 5 km (3.1 miles) of the fault. The surface rupture associated with this earthquake is thought to have extended for approximately 30 km (18.6 miles), from Warm Springs to San Leandro, with a maximum reported displacement of 1 meter (3.28 feet). As well as moving during earthquake ruptures, the Hayward fault also moves by aseismic creep. Measurements along the fault over the last two decades show that the creep rate is 5 to 9 mm/year (0.20 to 0.35 inch/year) (Lienkaemper et al., 1991). The Working Group on Northern California Earthquake Potential (1996) assigns MCEs of M_w 6.9 and recurrence intervals of 210 years for both the northern and southern segments of the Hayward fault. Recent research of historical documents has led to the conclusion that an earthquake in 1836, previously thought to have occurred on the northern Hayward fault occurred elsewhere, thereby increasing the time since the last earthquake on this segment of the fault. Recent paleoseismic trenching along the northern Hayward fault indicates that the last surface rupturing earthquake along this part of the fault was sometime between 1626 and 1724 (Lienkaemper et al., 1997). This study also indicated that there are at least four surface rupturing earthquakes in the last 2,250 years.

2.2.3 Hayward Fault - Southeast Extension

The Hayward fault zone extends to the south of Milpitas, along the eastern margin of the Santa Clara Valley, as a broad zone of reverse and reverse-oblique faulting. This zone of faulting includes the Quimby, Crosley, and Evergreen faults, as well as the southeastern extension of the Hayward fault. The predominant sense of motion on these faults is right-lateral strike-slip on the continuation of the Hayward fault itself, becoming reverse-

oblique, with an increasing thrust faulting component towards the west. The thrust faults dip to the northeast. Trenching investigations and the geomorphic expression of these faults indicate that they have been active during late Pleistocene, and possibly even Holocene time. The Working Group on Northern California Earthquake Potential (1996) assigns a maximum earthquake of M_w 6.5 to the southeastern extension of the Hayward fault.

2.2.4 Calaveras Fault

The Calaveras fault is a main component of the San Andreas system, branching off the main San Andreas fault south of Hollister, and extending northwards for approximately 120 km (75 miles) to die out in the area of Danville. The predominant sense of motion on the Calaveras fault is right lateral strike-slip. A smaller component of vertical displacement is evident in some areas along the fault trace. The Calaveras can be divided into two distinct sections, northern and southern, with the boundary located at Calaveras Reservoir.

The Calaveras fault has generated a number of moderate-size earthquakes in historic time, including: (1) the 1861 M_L 5.9 event; (2) the 1886 M_L 5.4 event; (3) the 1897 M_L 6.2 event; (4) a probable M_L 6.5 event in 1911; (5) the 1988 M_L 5.1 Alum Rock event; (6) the 1979 M_L 5.9 Coyote Lake event; and the 1984 M_L 6.2 Morgan Hill event.

To the south of Calaveras reservoir, microseismicity clearly delineates the active trace of the fault. The long term slip rate and contemporary creep rate for the southern Calaveras fault are approximately 15 mm/year (0.59 inch/year) (WGNCEP, 1996). The Working Group for Northern California Earthquake Potential (1996) calculated a MCE of M_w 6½ for the (southern and northern) Calaveras fault.

2.2.5 Monte Vista - Shannon Fault

The Monte Vista-Shannon fault zone is one of a series of southwest-dipping thrust and reverse-oblique faults along the southwestern margin of the Santa Clara Valley. Collectively known as the Foothills Thrust System, this is a series of range-front faults that mark the boundary between the southern Santa Cruz Mountains and the western Santa Clara Valley. The Monte Vista fault extends from Palo Alto, southeast, through Los Altos, Los Altos Hills and Cupertino to Saratoga. Rather than a single fault, this is a belt of southwest-dipping thrust faults. Limited trench data, stream channel profiles, and stream terrace elevations all indicate late Pleistocene and possibly Holocene movement on the Monte Vista thrust (Hitchcock *et al.*, 1994). To the south of Saratoga, the continuation of the Monte Vista fault is mapped as the Shannon fault. The Shannon fault also comprises a series of southwest-dipping thrust faults that offset Miocene bedrock and cut a paleosol reported to be less than 20,000-years-old [R. McLaughlin, U.S. Geological Survey, reported in Hitchcock *et al.*, (1994)]. The Monte Vista-Shannon fault is capable of generating a MCE of M_w 6¾ (WGNCEP, 1996).

2.2.6 Sargent - Berrocal Fault

The Sargent-Berrocal fault is a 94-km-long (58.4 miles), complexly branching system of southwest-dipping thrust and reverse oblique faults associated with the San Andreas fault. Evidence of recent movement along the Sargent fault include reports of surface rupture along the fault trace in the vicinity of Page Mill Road in Palo Alto during the 1906 earthquake and some possible aftershocks of the 1989 Loma Prieta earthquake. The northern two-thirds of the fault converges with the San Andreas fault at depth, and may not be an independent seismic source (WGNCEP, 1996). The Sargent-Berrocal fault is considered capable of generating a MCE of M_w 6¾.

2.3 GROUND MOTIONS

To characterize the attenuation of ground motions from the seismic sources (described above) to the Guadalupe River Flood Control Improvement Program, we have used empirical attenuation relationships appropriate for deep soil. These include Abrahamson and Silva (1997), Boore et al. (1993), Campbell (1997), and Sadigh et al. (1997). Eight 84th percentile peak horizontal ground accelerations (PGAs) were calculated for each of the designated four segments of the Guadalupe River Flood Control Improvement Project (Tables 3a to 3d). The limits of the four segments are summarized in the following table.

Segment	From	To
1	Alviso Marina	Tasman Drive
2	Tasman Drive	Montague Expressway
3	Montague Expressway	Highway 101
4	Highway 101	I-880

The PGAs were calculated using the MCE for each seismic source described above, and the closest approach of each of these sources to each reach of the project. For each site, the controlling seismic source is a M_w 6.5 earthquake along the southeast extension of the Hayward fault. This event is capable of generating a 84th percentile peak horizontal ground acceleration of 0.55 g in Segments 1 through 4, which include the levee limits of Task 2.4.5.

2.4 SUMMARY

There are no known active faults that cross the project alignment. However, the levees will be subject to strong ground shaking from known nearby active faults.

Section 3

3.1 SURFACE CONDITIONS

3.1.1 Field Reconnaissance

The surface conditions will be described for both the East and West Levees, starting at the north end (Station 8+000) at Route 237 and proceeding to the south end (Station 17+600) near I-880. This description is based on our previous field reconnaissance during which we walked the entire alignment of both levees on December 19, 22 and 23, 1997. At that time, we observed surface conditions of the crest, both side slopes and the inboard and outboard toe of each levee. Our observations were documented in detail and are presented in Appendix D of the April 2000 report. This documentation also included areas of apparent distress such as surface cracking, wet areas, settlement, deformation and undermining of the levee toe. At the time of our field visit, the water flowing in the Guadalupe River was confined to the low flow channel; therefore, the water surface generally was below the level of the inboard toe.

3.1.2 General

In general, the existing East and West Levees are of earthfill construction, are trapezoidal in cross section and range in height from about 2 ½ to 4 ½ meters (8 to 15 feet). The crest serves as an access road and is covered with armoring aggregate (gravel). In 1995 both levees between the SPRR and Highway 101 were raised, including some areas with soil reinforcement or concrete floodwalls; in many cases the top slopes were steepened. The entire 1995 crest was surfaced with armoring aggregate; the crest width is about 4 to 5 ¾ meters (13 to 14 feet). Numerous culverts and pipes also cross or penetrate the levees. Typically both the inboard and outboard slopes are vegetated with grass, weeds and scattered shrubs. Animal burrows are common along the entire alignment but are randomly located on the face of both side slopes. Access for vehicles is provided by the roadway on the crest and supplemented with ramps cut into the inboard slopes of both levees; at some locations an access road is also positioned just beyond the toe of the inboard slope. The low flow channel in some areas approximates the design centerline and in some areas meanders in close proximity to the inboard toe of the levee. Between the inboard toes of the levees, (i.e., in the channel area), the vegetation typically consists of weeds, grass and occasional trees.

Surface conditions for each segment are described in more detail in the following sections; refer to Appendix C of the April 2000 report for a more complete list of observations, including culverts and pipes observed during a site reconnaissance. For reference purposes for each segment, please refer to the topographic drawings in Figures 3a to 3o of the April 2000 report that include aerial photography.

3.1.3 Segment from Route 237 to Tasman Drive-Station 8+050 to Station 9+250

A bridge spans the river at Tasman Drive; the approach abutments are positioned on both levees.

East Levee

Just south of Route 237, an undeveloped area extends to about Station 8+150; within this area at Station 8+100 we observed on the outboard slope:

- an eroded gully/excavation in the upper slope; and
- a pit about 1 meter (3.3 feet) wide and 0.5 meter (1.6 feet) deep near the bottom of the slope, apparently manmade.

Beyond the toe of the outboard slope at this same location two manholes protrude above the ground surface.

From Station 8+150 to Station 8+630 a ditch parallels the outboard toe; beyond the ditch is a mobile home park. The bottom of the ditch is wet at Station 8+200 and between Stations 8+250 and 8+400. Within the mobile park is a large pond between approximately Station 8+340 to Station 8+450. At about Station 8+575 an eroded bike/foot path was observed on the outboard slope; an apparent erosion control mat had been displaced.

The Oakmead Pump Station is located beyond the outboard levee toe at Station 8+630. The outfall for this pump station consists of a concrete structure, as well as sacked concrete slope protection (SCSP) on the inboard slope of the levee. Beyond the toe and SCSP, the bottom of the channel is lined with riprap.

From Station 8+700 to Tasman Drive (Station 9+250), the adjacent property is presently under construction and appears to consist of 3-story apartments/condominiums with recreational facilities (tennis courts and swimming pool). The outboard toe of the levee was excavated to a height of about 0.5 meter (1.6 feet) and stockpiled with loose soil from about Station 8+700 to 9+150.

In close proximity to Tasman Drive (between Stations 9+200 to 9+230) the toe of the outboard slope was undermined; it appeared to be the result of an excavation for a recent pipeline installation.

Scattered animal burrows were observed along this levee.

West Levee

A storm water detention pond and pump station are positioned near Station 8+100 just beyond the toe of the outboard levee. An outfall on the inboard slope consists of three 24-inch pipes with flap valves which discharge water into the river channel.

Beginning at about Station 8+130 and continuing to Tasman Drive, a drainage ditch parallels the outboard toe at distances ranging from about 3 to 6 meters (10 to 20 feet); standing water was observed from the detention pond to about Station 8+700. Beyond the drainage ditch, the Santa Clara Golf Course is positioned on a relatively high old landfill area.

From Station 8+950 to Tasman Drive and beyond the ditch, abutting property is relatively flat and contains 1-story industrial/office buildings.

Along the levee we observed:

- A discontinuous tension crack on the upper inboard slope between Stations 8+200 to 9+150.
- Some settlement of the levee crest near the inboard hinge point between Stations 8+400 to 9+230.
- These two observations appear to be related to over-steepening of the raised fill (from 1995) along the inboard hinge point.

Just north of Tasman Drive, a construction pit for a pipeline under construction was noted.

In addition to scattered animal burrows throughout, a cluster of burrows was noted on the inboard slope near Station 9+060.

3.1.4 Segment from Tasman Drive to Montague Expressway-Station 9+250 to Station 11+820

A 6-lane wide bridge spans the river at Montague Expressway; the approach abutments are positioned on both levees.

Two buried water pipelines cross under the river at about Station 10+130. These include San Francisco Water Department Bay Division Pipeline Numbers 3 and 4 with diameters of 1.85 meters (73 inches) and 2.32 meters (91 1/2 inches), respectively.

East Levee

With the exception of two areas, the property abutting the East Levee between Tasman Drive and Montague Expressway contains 2-story high technology buildings and parking lots.

One area of exception includes the storm-water detention pond located between Stations 10+700 and 10+950 and about 5.4 meters (18 feet) beyond the toe of the outboard levee; it is at least 2.5 meters (8.2 feet) deep. It was dry at the time of our reconnaissance. An inlet structure for the pond is positioned near Station 10+970.

Another area of exception is undeveloped and located between Stations 11+500 and 11+820 (Montague Expressway).

Along the levee, we observed:

- A concrete floodwall with an L-shape cross section, is located along the outboard hinge point on the crest between approximate Stations 9+660 to 11+670
  - Several animal burrows extend under the floodwall at about Stations 9+550 and 9+880.

  - Near about Stations 10+120 and 10+200 a slight separation between the wall and backfill was noted.
  - Localized vertical hairline cracks.
- Between Stations 11+180 to 11+200 there is undermining to a height of about 0.6 meter (2 feet) at the inboard toe.
- Longitudinal tension cracks on the crest and near the crest on the outboard slope.

In addition to scattered animal burrows throughout, clusters were noted for most of this segment. In particular, numerous clusters of burrows on the outboard slope were noted between Stations 10+700 and 10+950; this coincides with the limits of the stormwater detention pond.

West Levee

The property abutting the West Levee is mostly undeveloped, with three exceptions. First, a small pond with standing water is located at about Station 9+280; it apparently controls storm flow as it contains several pipes and a concrete flow structure. Second, a storm water pump station is positioned near Station 10+160 beyond the outboard toe; 4 pipes penetrate the levee and discharge water into a concrete outfall structure at the toe of the inboard levee.

A third exception is the apartment complex located between Stations 10+900 to 11+600. These structures typically are 3 stories in height and contain a basement garage. A number of those buildings abut the outboard toe of the levee. One parking lot and several tennis courts are in close proximity to the outboard levee toe.

Along the levee we observed:

- Surface cracks from toe to crest at Station 9+750 on inboard slope.
- A steep side slope on outboard face between about Stations 9+755 to 9+850, and in particular the upper 3 meters (10 feet) between Stations 9+780 to 9+830.
- An 8-meter (26-foot) long crack at Station 10+100 on the outboard slope near the crest.
- Face of outboard slope between Station 10+400 to 10+470 was scraped off; area extended from toe to 1 meter (3.3 feet) below crest.
- A 305 mm (12-inch) apparent outfall at midheight of levee on inboard face at Station 10+600. Inlet point is unknown.
- Erosion at toe of inboard slope (apparent scour) at Station 10+900 (about 13 meters or 42 feet long) and Station 10+990 (about 20 meters or 66 feet long).
- A replaced section of levee between Stations 11+160 and 11+200 where seepage from a well had been observed in the fall of 1997. No seepage was observed during our field reconnaissance.
- A 60-meter (187-foot) long longitudinal crack between Stations 11+660 and 11+720 and another between Stations 11+745 and 11+790; both cracks are located near the top of the outboard slope.
- Two cracks on the inboard slope, one at Station 11+500 and another near Station 11+700.

3.1.5 Segment from Montague Expressway to Trimble Road-Station 11+820 to Station 13+700

East Levee

The property abutting the East Levee is developed and generally consists of 1 or 2-story high technology buildings and parking lots. An above grade water tank and associated facilities are located at Station 13+420 near Trimble Road.

Along the levee, we observed:

- Two different slopes on the inboard face between Stations 12+000 and 12+200 with a steeper apparent face at the toe. From Station 12+200 and 12+220, the toe was undercut about 0.3 meter (1 foot).
- A series of undermined local sections at the inboard toe associated with either erosion or slip failures. Maximum scarp height is 1 meter (3.3 feet) at these approximate locations.
  - Station 12+480, length 8 meters (26 feet)
  - Station 12+500, length 20 meters (66 feet)
  - Station 12+540, length 8 meters (26 feet)
  - Station 12+630, length 10 meters (33 feet)
  - Station 12+660 to 12+700
  - Station 13+000 to 13+100 (previous 1995 problem?)
  - Station 13+110, length 6 meters (20 feet)
  - Station 13+160, length 15 meters (49 feet)
- Three-meter (10-foot) long longitudinal crack at Station 13+320 near top of outboard slope.

West Levee

Along this portion of the West Levee, the abutting property has two types of developments:

- Single family residences between Station 11+840 (Montague Expressway) and 12+470; and
- Light industrial/warehouse type 1 and 2-story buildings between Stations 12+470 and 13+700 (Trimble Road), including numerous parking lots and a roadway near Station 13+300.

Along the levee we observed:

- From Station 11+950 to 12+450 there is a 1-meter (3-foot) drop off forming a shelf at the inboard toe.
- At Station 12+280 an access stairway is cut into the outboard face of the levee.
- Bulge at toe of outboard slope at about Station 12+750 and 12+860.
- Tire rutting on the gravel surface of the crest near Station 12+800.
- A 6-meter (20-foot) long longitudinal crack at Station 12+850 near top of inboard slope.

3.1.6 Segment from Trimble Road to Highway 101-Station 13+700 to Station 14+550

A 4-lane wide bridge spans the river at Trimble Road; the approach abutments are positioned on both levees. At Highway 101, an 8-lane wide bridge spans the river; its approach abutments are positioned on both levees.

East Levee

A park abuts the adjoining property from Trimble Road to about Station 14+050. Between Station 14+050 to Highway 101, the property is undeveloped and most of it is under cultivation. Several electrical transmission towers are located in this area.

Along the levee, we observed:

- Deep gully at Station 13+700 on slope at the northeast abutment of Trimble Road Bridge; it is located below the bridge drain outlet.
- Bulge at toe of inboard slope at Station 14+180 (7 meters or 23 feet long) and Station 14+200 (4 meters or 12 feet long).
- Shrinkage cracks on outboard face between Station 14+100 to 14+140 and between Station 14+200 to 14+350.
- Concrete slab serves as slope protection on inboard slope between Stations 14+350 to 14+500 (Highway 101). Two of 8 weepholes plugged.
- Scour at toe of inboard slope between Stations 14+225 to 14+330.

West Levee

The property abutting this levee is developed with light industrial buildings and parking lots from Trimble Road to about Station 14+010. From that station to Highway 101, the adjacent property is open, undeveloped and relatively flat.

Along the levee, we observed:

- Shrinkage cracks along the inboard and outboard slopes between Stations 13+800 to 14+270 and on crest between Stations 14+100 to 14+270.
- Tire ruts in gravel surface on crest between Stations 13+910 to 13+940 and between Stations 14+100 to 14+270.
- A 1,829-mm (72-inch) reinforced concrete pipe at Station 14+010 has a concrete U-type endwall outfall. Cracking was observed on the exposed invert.
- Steep inboard side slope between Stations 14+420 to 14+450.

3.1.7 Segment from Route 101 to I-880-Station 14+550 to Station 15+600 ±

Within this segment there are three bridges over the Guadalupe River including one at Airport Island (2 lanes), one at Brokaw Road (5 lanes) and one at Skyport Boulevard. At each bridge the abutments are positioned on both levees.

East Levee

Between Highway 101 and Station 15+230, a large parking lot for the San Jose International Airport (SJIA) abuts the levee. From Station 15+230 the East Levee

parallels Highway 87 (Guadalupe Parkway) to the end of the project at about Station 15+370.

Along the levee, we observed:

- The inboard slope underneath Highway 101 Bridge and beyond to about Station 14+580 is covered with a combination of concrete sacks and rough concrete.
- At about Station 14+660 the inboard slope above a 660 mm (26-inch) flap gate outfall is covered with concrete sacks and rough concrete adjacent to the south edge of the concrete sacks.
- The inboard slope under the east abutment of Airport Island Bridge is covered with gabion revetment.
- The levee is either small or non-existent, i.e. the channel is in a cut.
- Scour at the inboard toe between Stations 14+710 and 14+725 with a 0.3-meter (1-foot) drop.
- Between Stations 14+780 and 14+890 on inboard slope
  - old access ramp with steep side slope
  - average 1 meter (3 foot) drop at toe and a maximum drop of 1.5 meters (5 feet) at Station 14+800
- Slope change at inboard toe with a 1-meter (3-foot) drop between Stations 14+980 and 14+990.
- A 0.6-meter (2-foot) drop at inboard toe between Station 15+100 to 15+300

West Levee

Airport Boulevard parallels the west levee for the entire length of this segment; it varies in width from 2 to 4 traffic lanes. San Jose International Airport abuts this street and consists of parking lots, parking structures, terminal buildings, aircraft hangers and commercial structures. The levee in this segment is either small or non-existent or the channel is in a cut.

Along the levee, we observed:

- The inboard slope underneath Highway 101 Bridge and beyond to about Station 14+570 is covered with a combination of concrete sacks and rough concrete.
- Some of the curved inboard slope near Station 14+600 is covered with sacked concrete slope protection.
- The inboard slope under the west abutment of Airport Island Bridge is covered with a gabion revetment.
- A concrete slab covers the inboard slope from the edge of the gabion revetment to about Station 14+730. There is slight undermining of the toe of concrete slab.
- The lower part of the slope on the inboard face appears to steepen at a number of locations near Stations 14+820, 15+130, 15+360, 15+500, between 15+590 to 15+630, between 15+700 to 15+770, 15+780, between 16+890 to 17+130, between 17+340 to 17+530 (very steep face in lower 6 meters or 20 feet) and 17+570.

- A flattened inboard slope, recently regraded for the Skyport Drive Bridge now under construction, between Stations 16+160 to 16+240. Concrete abutments and bents were in place.
- Sporadic (random) concrete rubble fill (pieces and slabs) on face of inboard slope between approximately Stations 16+200 to 16+650 and 16+800 to 17+500.
- Cracking on adjacent asphalt concrete pavement on Airport Boulevard between Stations 15+090 to 15+360. However, no separation was noted between pavement and concrete curb and gutter.
- Ramps for access road cut into inboard slope with tension crack and slip between Station 15+700 to 15+770; another crack near Station 16+470.
- Steep face on inboard slope between Stations 16+890 to 17+130; possible scarp or eroded face.

3.2 SUBSURFACE CONDITIONS

3.2.1 Field Exploration

The subsurface conditions along the East and West Levees were investigated for the 2000 study by drilling 34 exploratory borings (Borings EB-01 through EB-17 and Borings WB-01 through WB-17) and 28 cone penetration tests (CPT) (EC-01 through EC-13 and WC-01 through WC-15) at the locations shown on Figure 2. The boring logs and CPT logs from the previous investigation were presented in Appendix E and Appendix F in the April 2000 report.

Between the limits of the current task, 5 additional borings (WB-21 through WB-25) were drilled through the West Levee embankment, 7 additional borings (EB-23 through EB-26 and EB-29 through EB-31) were drilled through the East Levee Embankment and 9 borings (CB-05 through CB-13) were drilled in the channel. Four (4) additional borings (EB-21, EB-22, CB-03 and CB-04 were drilled as part of the Route 237 bridge study (Task 2.4.1 and Task 2.4.2). A total of 22 additional borings were drilled to date. Please note that borings drilled on the East Levee start with the letter E; similarly borings on the West Levee start with the letter W, and borings within the channel start with the letter C. A detailed discussion of the techniques used for the subsurface investigation is presented in Appendix A. The depths of these borings ranged from 10.2 to 40.1 meters (33½ to 131½ feet). Figure A-1 presents the Unified Soil Classification System, as well as guidelines summarizing soil consistency and relative density used in preparation of the boring logs. Figure A-2 is a Boring Log Legend that includes the notation used for the types of samples and methods of advancing them. Comprehensive descriptions of the soils encountered at each location are presented on the Logs of Borings, Figures A-3 through A-26. The soil conditions encountered are summarized in Section 3.2.3 "Soil Conditions".

3.2.2 Laboratory Testing

Soil samples were carefully sealed in the field and returned to our laboratory for testing. Soil classifications made in the field were verified in the laboratory after further

examination and testing. Laboratory tests were performed on selected soil samples. These tests include, but are not limited to, water content, dry density and unconfined compressive strength. The results of these tests are presented at the corresponding sample locations on the Log of Borings. Additional tests including Atterberg Limits, sieve analysis, consolidation, and triaxial compression were made to better characterize the soil conditions and to estimate the compressibility and shear strength of the levee fill, foundation material and channel material.

Five (5) samples were selected for consolidation tests based on their quality and soil conditions encountered in the corresponding boring; samples tested include EB-23-8 (depth of about 9.5 m or 31 feet), EB-27-5 (depth of about 5.8 m or 19 feet), EB-27-9 (depth of about 13.1 m or 43 feet), WB-21-6 (depth of about 7.9 m or 26 feet) and WB-24-6 (depth of about 7.3 m or 24 feet). Undrained unconsolidated triaxial tests were conducted on 6 relatively undisturbed samples to better estimate their undrained shear strengths; samples tested include EB-23-4 (depth of about 4.9 m or 16 feet), EB-23-10 (depth of about 12.8 m or 42 feet), EB-24-8 (depth of about 11.0 m or 36 feet), EB-26-6 (depth of about 6.4 m or 21 feet), WB-21-9 (depth of about 13.1 m or 43 feet), and WB-22-8 (depth of about 9.5 m or 31 feet). A series of 4 consolidated undrained triaxial tests were conducted on 4 relatively undisturbed samples to simulate their shear strength behavior in the laboratory; samples tested include EB-24-4 (depth of about 5.2 m or 17 feet), WB-21-4 (depth of about 4.3 m or 14 feet), WB-25-6 (depth of about 8.2 m or 27 feet) and CB-07-7 (depth of about 9.8 m or 32 feet). Consequently a total of 12 stress-strain plots were produced, resulting in 12 Mohr's circles. Fifteen (15) sieve analyses were conducted to determine the grain size distribution of selected sand and gravel samples. Ten (10) Atterberg Limits tests were performed on selected fine-grained soils.

A more comprehensive discussion of the laboratory testing program is presented in Appendix B.

3.2.3 Soil and Groundwater Conditions

We have based our assessment of the soil conditions on two major sources:

- Field exploration conducted for this study (detailed logs of borings are presented in Appendix A); and
- Existing subsurface data from previous investigations by our predecessor firms as well as by others (a list of the sources and boring logs are presented in Table 2).

Ground surface elevations of borings were estimated from nearby cross section geometry provided by the District and from spot elevations provided by CH2MHILL.

For convenience of discussion, the subsurface soil and groundwater conditions along both the East and West Levees are presented below with reference to the stations of the control line as shown on Figure 2. Furthermore, we previously prepared two color-coded idealized soil profiles illustrating the subsurface conditions encountered in the 1999 and preceding investigations. These profiles are presented in Figure 5 and Figure 6 for the East Levee and West Levee, respectively, in our April 28, 2000 report. Based on those two figures and logs of twenty-three (21) 2001 and 2002 borings, we have prepared the

following description of the soil and groundwater conditions. Although these profiles display subsurface conditions along the entire alignment, the discussion is limited to the area between Route 237 and I-880.

East Levee

Stations 8+000 to 8+120

Levee fill extends to a depth of about 4.5 meters (15 feet). The fill consists of lean to fat clay which contains some sandy lenses and occasional wood fragments. These levee fills are very stiff to hard in consistency, conceivably due to method of placement and desiccation.

Below the levee fill, fat clay extends as deep as 11 meters (36 feet) and consists of Bay Mud with low density and high moisture content. Seams and lenses of fine sand were found throughout this layer.

Other foundation soils consist of lean clay with sand extending to 12.2 meters (40 feet). The clay is medium to stiff in consistency; it is moderately compressible. Underneath this clay layer is a sand deposit extending beyond the depth of most of the previous borings. The sand is medium dense to dense in relative density, and is relatively incompressible.

Groundwater levels measured in the borings for the past 30 years appear to reflect seasonal changes between Elevation -1.2 to 1.5 meters (-4 to 5 feet). In the 2000 investigation, groundwater was encountered in EB-01 at approximately Elevation -2.1 meters (-7 feet).

Stations 8+120 to 9+550

Levee fill 4.6 to 5.5 meters deep (15 to 18 feet) extends generally to about Elevation 0.3 meter (1 foot). The fill consists primarily of lean to fat clay with some gravel, exhibiting very stiff to hard consistency. Below the fill, an upper lean clay extending to approximate Elevation -3.7 meters (-12 feet) is medium to stiff in consistency. Underneath this upper lean clay, a layer of lean to fat clay was found to extend to about Elevation -7.0 to -7.6 meters (-23 to -25 feet), and is soft in consistency. Below the clay strata, a deposit of dense to very dense silty sand to poorly graded sand and gravel occurs from Elevation -7.0 to -14.9 meters (-23 to -49 feet). Below the granular deposits, stiff lean clay with some sand lenses extends beyond the terminal depth of borings at about Elevation -25.3 meters (-83 feet).

Groundwater levels encountered in the 2000 borings showed variations between Elevations -1.5 and 0.9 meters (-5 and 3 feet). The fluctuation probably reflects the seasonal changes in the river levels.

Stations 9+550 to 12+450

Levee fill consisting of stiff to hard lean clay about 3.0 to 4.3 meters (10 to 14 feet) thick extends to approximately Elevation 2.4 meters (8 feet). The foundation soils consist of lean clay and interbedded sand layers extending to the terminal depths of most of the borings in this reach, at approximately Elevation -6.7 meters (-22 feet). The lean clay stratum also extends to the terminal depth of deeper boring HHP-1 at approximately Elevation -28.0 meters (-92 feet). The clay layers above Elevation -1.8 meters (-6 feet) are soft to medium in consistency and are considered to be moderately to highly compressible. Below Elevation -1.8 meters (-6 feet), the clays are medium to stiff in consistency and moderately compressible. The interbedded sand layers are generally dense to very dense with thickness varying from 0.9 to 3.4 meters (3 to 11 feet). However, in Boring GRIV-8 the sand layer is very loose in relative density.

Groundwater was not encountered in the 2000 investigation. However, in a previous study (WCC 1992) conducted between January and February 1992, groundwater levels were noted at Elevations 0.5 to 1.5 meters (1.5 to 5 feet). It was concluded that the rise and fall of the river level had no apparent direct communication to the groundwater level encountered in the wells installed.

Stations 12+450 to 15+600

Levee fill was found to consist of lean clay with interbedded sand layers; the thickness varies from about 3.0 to 1.8 meters (10 to 6 feet) and becomes thinner towards the southern end of the channel alignment. The bottom of the levee fill varies from approximately Elevation 4.6 to 7.6 meters (15 to 25 feet). The fill material is very stiff to hard in consistency and is incompressible.

Below the fill, a stratum of lean clay with variable thickness extends to Elevations 0 ± to 6.1 meters (0 ± to 20 feet) towards the southern end of the channel alignment. This clay is medium to stiff in consistency and is considered moderately compressible. Underneath this clay stratum, dense silty sand deposits of variable thickness from 0.9 to 3.0 meters (3 to 10 feet) extend to Elevation 1.8 to 3.0 meters (-6 to 10 feet) towards the southern end of the channel alignment. Another layer of lean clay with consistency ranging from stiff to very stiff occurs below the upper sand and extends to approximately Elevation -3.7 meters (-12 feet). In deeper borings drilled previously (WCC 1982), another layer of sand and gravel was encountered to Elevation -9.8 to -12.2 meters (-32 to -40 feet). This sand deposit is very dense and is considered low in compressibility.

Groundwater levels measured in the 2000 borings in this reach varied from Elevation 2.4 to 6.4 meters (8 to 21 feet), in a north to south direction. This water level appears to be consistent with the change in elevation of the upper granular layer.

West Levee

Station 8+000 to 8+500

A heterogeneous fill consisting of lean clay to silt forms the levee that extends to approximate Elevation -1.5 to 1.5 meters (-5 to 5 feet). The fill material is very stiff to hard in consistency within the upper 1.5 to 2.1 meters (5 to 7 feet) and becomes medium to soft near the fill and foundation soil interface. Between approximately Station 7+500 and 7+800, a mixture of soft, elastic silt to fat clay occurs below the levee fill and extends to approximately Elevation -9.1 to -10.4 meters (-30 to -34 feet). An old river meander occurred within this reach, so it is conceivable the silt and clay mixture are the "dredged mud" fill from the construction of the 1960 levee improvement project. At Boring WB-03, very soft, lean to fat clay was encountered between approximately Elevation -3.4 to -5.2 meters (-11 to -17 feet). The occurrence of this soft clay layer is consistent with the location of an old river meander; these deposits could be the fine sediments associated with the old river channel.

For the portion of this reach west of Station 7+500, the foundation soil below the levee fill, extending to Elevation -8.2 to -8.8 meters (-27 to -29 feet), consists of lean clay of medium to stiff consistency, and it is considered to be moderately compressible. Below the clay layer, deposits of medium dense silty sand extend beyond approximately Elevation -18.0 meters (-59 feet).

Groundwater levels measured in the 1960's exploration varied between Elevation -0.9 to 0.6 meter (-3 to 2 feet). During the 2000 investigation, no groundwater was encountered above approximate Elevation -1.2 meters (-4 feet).

Station 8+500 to 9+200

Levee fill consisting of a lean clay and silt mixture extends to approximately Elevation 0.9 to 0 meter (3 to 0 feet). The fill material exhibits very stiff to hard consistency, and it is considered to be low in compressibility. Below the fill, a lean clay layer extends to approximately Elevation -2.4 to -3.7 meters (-8 to -12 feet). This clay layer is medium to stiff in consistency and is considered moderate in compressibility. Underneath this clay layer lies a silty sand to poorly graded sand and gravel deposit that extends beyond the terminal depths of the borings within this reach at Elevation -7.6 to -12.2 meters (-25 to -40 feet).

Groundwater levels measured in the 1960's and 1980's investigations varied between Elevation -0.3 to 0± meters (-1 and 0± feet). Boring WB-04 from the 2000 investigation revealed no groundwater above Elevation 0.9 meter (3 feet).

Station 9+200 to 10+300

Levee fill extends to approximately Elevation 1.5 to 3.4 meters (5 to 11 feet). The fill consists of a lean to fat clay mixture of stiff to very stiff consistency; it is considered to be low in compressibility. Underneath the fill, in general a layer of fat clay overlies a

lean clay sublayer and clayey sand to poorly graded sand and gravel deposit. The fat clay extends to approximately Elevation -8.5 to -4.6 meters (-28 to -15 feet) towards the southern end of this reach; it is soft to medium in consistency. The lean clay sublayer occurs towards the southern end of the reach and is stiff in consistency. The granular deposits underneath the clays are medium dense to very dense and extend to approximately Elevation -13.7 to -15.2 meters (-45 to -50 feet). Deeper borings such as TDB-2 and HHP-4 revealed a lean clay deposit extending beyond Elevation -24.4 meters (-80 feet); the clay soil was medium to stiff in consistency.

Groundwater levels measured in the 1960's and 1980's investigations varied from Elevation 0 to -0.3 meter (0 to -1 foot). Boring WB-05 from the 2000 investigation encountered no groundwater above Elevation 0.8 meter (2½ feet).

Station 10+300 to 11+750

Levee fill varies from 4.0 to 5.2 meters (13 to 17 feet) in thickness in this reach and extends to approximately Elevation 1.5 to 3.0 meters (5 to 10 feet). The fill is stiff to hard in consistency in the upper 3.0 meters (10 feet); it becomes medium near the fill and foundation soil interface.

Underneath the fill lies a deposit of lean clay, with a sublayer of fat clay towards the northern end of the reach; it is a continuation from the preceding reach. The lean clay layer is medium to stiff in consistency, whereas the fat clay sublayer is soft to medium. The bottom of this clay layer varies between approximately Elevation -2.4 to -5.2 meters (-8 and -17 feet). A 0.6 to 0.9 meter (2 to 3-foot) thick, loose to medium dense silty sand layer occurs below the clay layer.

Groundwater levels measured in the 1960's investigation varied between approximately Elevation 2.1 and 3.7 meters (7 and 12 feet). Boring WB-06 from the 2000 investigation encountered no groundwater above approximately Elevation 2.1 meters (7 feet).

Station 11+750 to 14+000

Levee fill 4.6 to 5.5 meters (15 to 18 feet) in thickness extends to about Elevation 3.0 to 7.6 meters (10 to 25 feet); this fill thins to about 2.4 meters (8 feet) south of Trimble Road. The fill is stiff to hard in consistency in the upper 3.0 meters (10 feet); it becomes medium to stiff below approximately Elevation 4.6 meters (15 feet). Below the fill lies a layer of lean clay where the bottom extends to approximate Elevation -1.5 to 4.6 meters (-5 to 15 feet); this clay is medium to stiff in consistency. A deposit of silty sand underlies the clay; the sand is medium dense to dense. Below the sand is a medium to very stiff lean clay extending to approximately Elevation -4.9 meters (-16 feet).

First groundwater encountered during the 2000 field exploration varied between approximately Elevation 4.0 and 4.6 meters (13 and 15 feet). Stabilized groundwater was then measured at approximately Elevation 5.8 meters (19 feet).

Station 14+000 to 15+550

Levee fill consisting primarily of lean clay extends to approximately Elevation 8.2 to 10.1 meters (27 to 33 feet). The clayey fill is very stiff to hard in consistency. Underneath the fill is a layer of lean clay which extends to approximately Elevation 6.1 to 3.0 meters (20 to 10 feet). The clay is soft to medium in consistency. Underlying the clay is a sand to silty sand deposit extending to approximately Elevation 1.5 to 4.6 meters (5 to 15 feet); the sandy deposit is medium dense. Another lean clay layer occurs below the sandy deposit and extends beyond the terminal depths of borings within this reach at approximately Elevation -1.5 meters (-5 feet). The clay is medium in consistency.

Groundwater levels measured in the 1980s varied between Elevation 5.5 and 6.7 meters (18 and 22 feet); the stabilized water level was at about Elevation 9.5 meters (31 feet) in Boring SJIA-2. During the 2000 investigation, no groundwater was encountered above Elevation 7.3 meters (24 feet) in Boring WB-14.

Station 15+550 to 15+800

About 3.0 meters (10 feet) of levee fill was encountered to approximately Elevation 10.7 meters (35 feet). The fill consists of very stiff lean clay. The foundation soils consist of highly variable interbedded layers of clays and sands. In the south half of this segment, a 2.1 to 4.3-meter (7 to 14-foot) thick layer of stiff to hard fat clay underlies the fill. A stiff lean clay layer underlies the fill and fat clay and extends to between approximately Elevation 6.1 and 3.0 meters (20 and 10 feet). It is considered to be moderate in compressibility. About 0.9 to 3.7 meters (3 to 12 feet) of silty sand underlies the lean clay and extends to approximately Elevation 0.6 to 3.0 meters (2 to 10 feet). The sand is medium dense. A soft to stiff lean clay layer underlies the sand and extends to approximately Elevation -5.5 to 1.2 meters (-18 to 4 feet). Another deposit of sand occurs below the lean clay and extends to approximately Elevation -4.6 to -9.4 meters (-15 to -31 feet); the sand is dense to very dense. Underneath the sand deposit is a lean clay layer that extends beyond the terminal depth of Boring SJIA-21 in this reach to approximately Elevation -12.8 meters (-42 feet).

Groundwater levels measured in Borings WB-15 and SJIA-S2 varied between Elevation 7.0 and 7.3 meters (23 and 24 feet). Stabilized groundwater levels measured in Boring SJIA-16 and SJIA-18 varied between Elevation 9.1 and 9.5 meters (30 and 31 feet).

Station 15+800 to 16+750

Levee fill about 2.4 to 3.0 meters (8 to 10 feet) thick extends to approximately Elevation 12.2 to 12.8 meters (40 to 42 feet). The fill consists of a lean and fat clay mixture stiff to very stiff in consistency. Underneath the fill, a layer of fat clay occurs to about Elevation 7.9 to 8.5 meters (26 to 28 feet); it is medium to stiff in consistency. Directly below the fat clay layer, foundation soils consist of a layer of lean clay and an underlying sand layer that extends beyond the terminal depths of borings within this reach. The sand stratum is at least 2.4 to 4.3 meters (8 to 14 feet) thick. The lean clays are soft to stiff in consistency, whereas the sand deposits are loose to medium dense.

Groundwater levels measured during the 2000 investigation in borings within this reach varied between approximately Elevation 8.8 and 10.1 meters (29 and 33 feet).

Station 16+750 to 17+500

Levee fill about 1.5 to 3.0 meters (5 to 10 feet) thick overlies lean clay strata with an interbed of medium dense silty sand. The fill extends to approximately Elevation 13.7 to 16.8 meters (45 to 55 feet) and is very stiff to hard. The lean clay deposit underneath the fill extends beyond the terminal depths of the borings within this reach; it is very stiff to hard in consistency to a depth of about 5.5 meters (18 feet) and is considered moderate in compressibility. Below this deposit, the lean clay layer is soft to medium. The silty sand interbed is about 0.6 to 1.5 meters (2 to 5 feet) thick and is positioned at about Elevation 12.5 to 13.1 meters (40 to 43 feet).

Groundwater levels measured during the 2000 investigation in nearby Boring WB-16 became stable at approximately Elevation 9.0 meters (29½ feet). Additionally, in Boring WB-17, no groundwater level was encountered above Elevation 12.2 meters (40 feet).

River Channel

Nine Borings, CB-05 through CB-13, were drilled in the river channel, i.e. between the inboard toes of the East Levee and West Levee for Task 2.4.5. These borings were performed to determine subsurface conditions for improvements at several bridges and for slope stability of the levees. Two additional borings, CB-03, and CB-04 were drilled in the river channel for the Route 237 Bridge study (Tasks 2.4.1 and 2.4.2). The locations of the borings are shown in Figure 2, Site and Field Exploration Location Plan. As shown in Figure 2, there are typically clusters of two borings, particularly at the bridge locations. The approximate locations of the six boring clusters are as follows:

Cluster Number	Channel Borings	Approximate Station (m)	Comment
1	CB-03, CB-04	8+100	South of Route 237 Bridge
2	CB-05, CB-06	9+300	South of Tasman Drive Bridge
3	CB-07, CB-08	11+800	Both sides of Montague Expressway Bridge
4	CB-09, CB-10	14+550 14+750	South of Highway 101 Bridge
5	CB-11	15+700	South of Airport Parkway Bridge
6	CB-12, CB-13	17+400	North of I-880 Bridge

In general, the channel borings show fill underlain by highly variable native alluvial soils. The highly variable native soils would be expected in the river channel. A more detailed discussion of the channel subsurface sediments at each cluster is presented below.

Cluster 1 Borings CB-03 and CB-04

Levee fill extends to depths between 2.4 and 4.0 meters (8 and 13 feet). The fill consists of lean to fat clay that contains gravel, wood fragments, and organics. These levee fills are soft to medium in consistency.

Below the levee fill, lean clay and silt are the predominant soil types. Lenses of sand and gravel up to 4.6 meters (15 feet) in thickness, but primarily less than 1.5 meters (5 feet) are present throughout. These granular deposits are medium dense to very dense; an exception is a loose silty sand layer in Boring CB-03 at a depth of 6.4 meters (21 feet).

Groundwater levels measured in CB-03 and CB-04 were at the ground surface, at approximately Elevation 1.2 to 1.5 meters (4 to 5 feet).

Cluster 2 Borings CB-05 and CB-06

Levee fill extends to depths between 2.4 and 4.6 meters (8 and 15 feet). The fill consists of layers of lean to fat clay and silty to clayey sand. The cohesive levee fill is medium to stiff in consistency. The granular levee fill is loose to medium dense.

Below the levee fill, lean to fat clay extends as deep as 9.1 meters (30 feet); it is soft to stiff in consistency. Silty sand was encountered in Boring CB-05 and CB-06 at approximately Elevation –7.3 meters (–24 feet), and extends to the terminal boring depth. This granular layer is dense in Boring CB-05 and loose in Boring CB-06.

Groundwater was measured in CB-06 were at approximately Elevation –2.0 meters (–6.5 feet).

Cluster 3 Borings CB-07 and CB-08

Levee fill extends to depths between 1.2 and 1.8 meters (4 and 6 feet). The fill consists of lean clay. The levee fill is stiff to very stiff in consistency. The granular levee fill is loose to medium dense.

Below the levee fill, lean clay extends to depths of 6.1 to 10.1 meters (20 to 33 feet); it is medium to stiff in Boring CB-07 and soft in Boring CB-08. Silty sand and silt were encountered in Boring CB-07 and CB-08 beneath the lean clay. These granular deposits are very dense (Boring CB-07) and loose (Boring CB-08).

Groundwater was measured in CB-08 were at approximately Elevation 0.7 meter (2.2 feet).

Cluster 4 Borings CB-09 and CB-10

Levee fill extends to a depth of 1.8 meters (6 feet) in Boring CB-09. The fill consists of layers of clayey gravel and lean clay. The levee fill is medium dense and stiff for the gravel and clay layers, respectively.

Below the levee fill in Boring CB-09, interbedded layers of lean clay, sand, and silt are present to the terminal boring depths. The clay is medium to stiff, and occasionally soft, whereas the sand is loose to medium dense. In Boring CB-10, soft to medium lean clay was encountered to the terminal boring depth, with the exception of a medium dense silty sand layer at Elevation 2.0 meters (6.6 feet).

Groundwater was measured in CB-09 and CB-10 at approximately Elevation 6.4 meters (21 feet).

Cluster 5 Boring CB-11

Levee fill extends to a depth of 3.5 meters (11.5 feet). The fill consists of clayey sand and gravel. The levee fill is loose to medium dense.

Below the levee fill, medium dense to dense silty sand and sand extends to a depth of 5.6 meters (18.5 feet). A lean clay layer was encountered at Elevation –0.8 meter (–2.5 feet) that extends to the terminal boring depth; it is medium in consistency.

Groundwater was measured in CB-11 at approximately Elevation 6.9 meters (22.5 feet).

Cluster 6 Borings CB-12 and CB-13

Borings CB-12 and CB-13 encountered interbedded layers of lean to fat clay and sand to the terminal boring depths. The cohesive layers are medium to very stiff in consistency. The granular layers are medium dense to very dense.

Groundwater was measured in CB-12 and CB-13 at Elevation 8.2 meters (27 feet) and 10.1 meters (33 feet), respectively.

Section 4

4.1 CROSS SECTIONS SELECTED FOR ANALYSIS

During our meeting of November 10, 1998 with the District, we presented two color-coded subsurface profiles (Figures 6 and 7 in the April 2000 report), one for the East Levee and a second for the West Levee. Our interpretation of the soil layers in each profile was based on our evaluation of the subsurface information, including both borings and cone penetration tests (CPTs). Based on these two profiles, we identified a total of 23 levee sections as candidates for analysis along the entire alignment between Station 6+200 and Station 17+500; the levee sections are listed in Table 4. At the request of the District for the 2000 study, we selected and performed engineering analyses for what we believe represented the seven (7) most critical cross sections within the 23 levee sections.

For this current study (Task 2.4.5), we have analyzed seventeen (17) additional cross sections between Station 8+000 and 17+500. A list of the seventeen (17) cross sections is presented in Table 5A and the location of each cross section in plan view is shown in Figure 2. The basis for selection of the seventeen (17) cross sections is presented below. In addition, we have analyzed 14 cross sections for a proposed side slope of 1.5H:1V on the outboard side of the levee. The approximate stations of these 1.5H:1V cross sections are presented in Table 5B and their locations are shown on Figure 2.

4.1.1 Selection of 23 Levee Cross Sections

The two idealized soil profiles (Figure 6 and Figure 7 of the April 28, 2000 report) presented in our November 10, 1998 meeting were based on the logs of 185 previous borings, as well as the 34 new borings and 28 new CPTs for the 2000 study. Based on our engineering judgment and interpretation of these two profiles, we identified 13 levee sections along the West Levee (Sections W1 through W13), and 10 levee sections along the East Levee (Sections E1 through E10). The approximate channel control line station limits of each levee section are listed in Table 4. We also plotted the limits of each levee section in plan view on Figure 2. Although a total of 23 levee sections for both West and East Levees were initially identified, in our opinion three (3) sections of the West Levee (Sections W2, W9 and W10) have similar subsurface conditions as the East Levee (Sections E2, E9 and E10, respectively). Therefore, from an engineering viewpoint, we believe the subsurface conditions along the entire project alignment can be represented by a reduced total of 20 levee sections. Twenty of the 23 previously identified levee sections lie within the limits of this study.

4.1.2 Selection of 17 Levee Cross Sections for Task 2.4.5

According to our Agreement, a maximum of seventeen (17) levee cross sections were to be analyzed. We have selected seventeen (17) sections that we believe are reasonably representative of the most critical conditions. Critical is defined as most likely having the lowest factor of safety for slope stability, based on our engineering judgment. The seventeen (17) levee cross sections we selected are shown in Table 5A and are defined as Cross Sections 14 through 30. The reasons for selecting these seventeen (17) are listed under the column entitled "comments" in Table 5A; they vary from the presence of loose

sand in the levee foundation, to steep side slopes, to levee raise, to sediment removal, to a ditch at the outboard toe. The location of each cross section is shown in plan view in Figure 2.

Initially, one critical cross section was chosen from each original levee section identified with similar soil conditions. For comparison, the average side slope inclinations and slope heights were tabulated. Selection of the critical cross section in each levee section was based on slope inclination and slope height. Slopes with steeper side slopes or larger heights were judged to have a lower factor of safety. Additional cross sections were then selected where deemed necessary by project scope or engineering judgment.

The levee improvements (as indicated by the plans prepared by CH2MHILL, dated April 2002) were incorporated into the selected critical cross sections. The proposed geometry, including levee raise and excavation beyond the inboard toe, was incorporated into the slope stability and seepage analyses. The design high water level was obtained for each cross section from the plans prepared by CH2MHILL.

Soil conditions were also a factor in the selection of the critical cross sections. Loose sand layers, for example, are considered critical. Locations of loose sand indicated on Figure 46 of the April 2000 report, and indicated by blow count data within the levee section, were also reviewed for selection.

We prepared two drawings for each of the seventeen (17) cross sections: (1) one drawing showing a cross section representing the soil conditions and existing levee geometry, and a table summarizing the selected geotechnical design parameters (Figures 5 through 21); and (2) a second drawing showing four graphs where the undrained shear strength (S_u), moisture content, dry unit weight and blows per foot are plotted with respect to elevation (Figures 22 through 38). The latter drawing includes data from all available borings in both levees within the station limits of the relevant levee section. The District furnished all original geometric levee sections; levee improvements were then incorporated based on the plans prepared by CH2MHILL.

4.1.3 1.5H:1V Cross-Sections

Because of limited right-of-way at the levee outboard slopes and the requirements of keeping the channel and inboard levee slopes at 2H:1V, CH2MHILL proposed to raise the levee by steepening the levee outboard slopes to 1.5H:1V at selected reaches. CH2MHILL provided fourteen (14) representative cross sections for such proposed levee reaches (Table 5B). For each of these 14 cross sections, a drawing was prepared showing the soil conditions and proposed levee geometry, and a table summarizing the selected geotechnical design parameters (Figures 47 through 60). On each of these figures, we have referenced the applicable figure showing Su, moisture content, dry unit weight and blows per foot (Figures 22 through 38).

4.1.4 Selection of Shear Strength Parameters

At the thirty-one (31) levee cross sections, shear strength parameters were selected for each soil layer based on the field and laboratory data. The selected design parameters are summarized in the upper left-hand corner table of Figures 5 through 21 for Cross

Sections 14 through 30, and Figures 47 through 60 for Cross Sections 39 through 44, 46, 47 and 50 through 55. These methods are described in more detail as follows.

Cohesive Soils in Levee and Levee Foundation

Undrained Strength

As an example, refer to Cross Section 14 on Figures 5 and 22 where four cohesive soil layers are shown in the levee fill and foundation including:

- Layer 1 is Lean Clay fill (CL), about 4.3 meters (14 feet) thick
- Layer 2 is Lean Clay (CL), about 4.9 meters (16 feet) thick
- Layer 3 is Fat Clay (CH), about 1.8 meters (6 feet) thick
- Layer 5 is Lean Clay (CL), about 1.8 meters (6 feet) thick

As shown in S_u versus elevation in Figure 22, all of the unconfined compression, undrained unconsolidated triaxial shear strength tests (UU) and CPT tests results within the limits of levee Section Number W3 are plotted. Based on this plot for Layer 1 (Elevation 5.5 to 1.2 meter or Elevation 18 to 4± feet) an S_u value of 63.2 kPa (kilo Pascals) [1,320 pounds per square foot (psf)] was selected as represented by the solid vertical line. For Layer 2 (Elevation 1.2 to -3.7 meters or Elevation 4 to -12± feet) an S_u value of 38.3 kPa (800 psf) was selected. For Layer 3 (Elevation -3.7 to -5.5 meters or Elevation -12 to -18± feet) an S_u value of 38.3 kPa (800 psf) was selected. For Layer 5 (Elevation -6.4 to -8.2 meters or Elevation -21 to -27± feet) an S_u value of 38.3 kPa (800 psf) was selected. These values were then entered in the table at the upper right-hand corner of Figure 5. It should be noted that the S_u value selected is not a lower bound; in some cases the S_u values plotted are believed to underestimate the soil strength because of the presence of sand interbeds in the samples tested.

In a similar manner, the moisture content and dry unit weight for each layer in Cross Section 14 were obtained from the other plots in Figure 22 and entered into the table in the upper right hand corner of Figure 5.

The embankment strengths were reduced to account for progressive failure in embankments on soft clay foundations. Since the embankment (fill) shear strengths were considered to be relatively high, the embankment strengths were reduced according to the procedures outlined by Duncan and Buchignani (1975). The plot used to determine the embankment strength reduction factor (R_E) is presented in Figure 39. For the foundation, we used a foundation strength reduction factor of 1.0 for Type A soil, also presented in Figure 39.

Effective Strength

The data from consolidated undrained (CU) triaxial shear strength tests with pore pressure measurements are presented in Table 6. These test results include both existing tests and new tests performed for this current investigation; they are divided into the following five categories:

- Lean clay fill (CL), dry unit weight < 17.28 kilo Newton per cubic meter (kN/m^3) [110 pounds per cubic foot (pcf)
- Lean clay fill (CL), dry unit weight > 17.28 kN/m^3 (110 pcf)
- Fat clay (CH), dry unit weight < 12.57 kN/m^3 (80 pcf)
- Lean clay (CL), 12.57 kN/m^3 (80 pcf) < dry unit weight < 15.71 kN/m^3 (100 pcf)
- Lean clay (CL), 15.71 kN/m^3 (100 pcf) < dry unit weight < 17.28 kN/m^3 (110 pcf)

Two effective strength parameters are shown in Table 6 including ϕ' (angle of internal friction) and c' (cohesion); values are listed for peak stress, 10 percent and 20 percent strain criteria. A design value was selected between the peak stress and 10 percent strain; in our judgment 20 percent strain is too large a deformation for the levee to undergo. The value from Table 6 was then entered into the upper right-hand corner table of Figure 5 for Layers 1 through 3 and 5. For example, Layer 1 was lean clay fill with a dry unit weight of 17.28 kN/m^3 (110 pcf); from Table 6 for a dry unit weight greater than 17.28 kN/m^3 (110 pcf), ϕ' = 33 degrees and c' = 0 kPa (0 psf) were selected.

Table 6 was prepared based on the graphical CU test results plotted in Figures 40 through 44. For example for the lean clay fill category with samples with dry unit weight less than 17.28 kN/m^3 (110 pcf) for peak stress, all q versus p values were plotted in Figure 40. The q and p values are defined as follows:

- $q = \frac{1}{2} (\sigma_1' - \sigma_3')$
- $p = \frac{1}{2} (\sigma_1' + \sigma_3')$

The best fit straight line (method of least squares) was then plotted in Figure 40 for peak stress and the ϕ' and c' values calculated (as shown in that figure) as 23.7 degrees and 21.8 kPa (456 psf), respectively.

Three-Stage Computation

To model the undrained loading due to sudden drawdown, we have adopted the simplified procedure proposed by Duncan, Wright and Wong (1990) to estimate the undrained shear strength at the potential failure plane. This procedure is similar to that originally recommended by Lowe and Karafiath (1960) with further simplification that linear interpolation is used to determine the effects of anisotropic consolidation. This 3-stage computation for cohesive soils is consistent with the guidelines for sudden drawdown analysis described in Corp Engineering Manual for Design and Construction of Levees (EM 1110-2-1913). Table 7 summarizes the corresponding sets of shear strength parameters (d_R, d_S, ψ_R, ψ_S) for the cohesive soils in the levee.

Cohesive Soil in Channel

Undrained Strength

Borings were advanced in the channel (beyond the inboard toe of the levee) as part of this study in order to assess the strength of the materials subjected to periods of wetting and

riverbed deposition. In general, the undrained strengths of the channel sediments were lower than undrained strengths of soils within the levee or levee foundation.

The extent of the channel sediments was estimated by extending a ½:1 (horizontal to vertical) line beneath the inboard levee toe. Cohesive soil inboard of this line was assumed to have channel sediment properties. Cohesive soil outboard of this line was assumed to have the properties determined from the index property plots of levee and foundation soils.

For Cross Sections 14 through 30, the dry unit weight, moisture content, and undrained strength of the channel sediments were estimated from the results of dry unit weight, moisture content, and unconfined compression tests performed on samples from Borings CB-03 through CB-13. The plots of undrained strength, moisture content and dry unit weight versus depth for the channel sediments are presented in Figures 22 through 38. Selected design parameters for the channel sediments are represented by a dashed vertical line and the values were inserted in the tables located in Figures 5 through 21. The selected undrained strength was estimated using the results of laboratory testing, local experience, and engineering judgment.

Effective Strength

The effective strengths of the cohesive channel sediments were determined in the same manner as outlined for cohesive soils in the levee and levee foundation.

Three-Stage Strength

Since the channel sediments and native clays are relatively soft and more or less normally consolidated, they do not exhibit undrained strength characteristics due to anisotrophy. Therefore, the effective strength is appropriate for the sudden drawdown condition.

Granular Soils

Effective strength parameters were estimated by the following method (refer to Layer 4, silty sand, in the upper right-hand corner table of Figure 5). The effective stress σ_v' at the middle of Layer 4 was calculated to be about 100.5 kPa (2,100 psf). For a Standard Penetration Test N value of 12 and σ_v' of 100.5 kPa (2,100 psf), the estimated relative density (D_r) is about 60 percent (see Figure 45). According to a published graph (see Figure 46) by NAVFAC (1982) for a silty sand and D_r of 60 percent, and an approximate dry unit weight of 105 pounds per cubic foot (pcf), ϕ' is about 34 degrees.

In conclusion, it is our opinion the seventeen sections selected and listed in Table 5A (Cross Sections 14 through 30) are reasonably representative of the subsurface conditions that may be critical from a levee stability and design viewpoint. We performed slope stability analyses at each of these sections based on the parameters summarized in Figures 5 through 21 for the existing levee geometry with the improvements as specified by CH2MHILL.

4.2 METHOD OF ANALYSIS

Four design conditions were analyzed for the slope stability evaluation in accordance with the Corps Manual for the Design and Construction of Levees (EM 1110-2-1913). The conditions, the slopes analyzed, shear strength guidelines, and minimum factors of safety required for design are listed below.

Case	Design Condition	Slope Analyzed	Required Minimum Factor of Safety
I	End of Construction	Riverside and Landside	1.3
II	Sudden Drawdown	Riverside	1.0
III	Steady Seepage from Full flood Stage	Riverside and Landside	1.4
IV	Earthquake	Riverside and Landside	Not applicable

As outlined in the Corps' guidelines, earthquake loadings are not considered in analyzing the stability of levees because of the low probability of earthquake coinciding with a period of high water. Nevertheless, we evaluated the seismic stability of the levee in the following manner because of the close proximity to major fault systems such as the San Anderas and Hayward:

- Performed pseudo-static slope stability analysis using a seismic coefficient approximately one half of the peak ground acceleration (0.3 to 0.33); this is consistent with the California Department of Conservation, Division of Mines and Geology, Special Publication 117, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," (1997).
- Performed simplified deformation analysis for cross-sections exhibiting a pseudo-static slope stability safety factor less than 1.0 (Makdisi and Seed, 1977) to estimate the magnitude of potential permanent deformation due to strong earthquake shaking.

The stability of the levees under Cases I, III and IV (pseudo-static) was evaluated using a limit equilibrium method based on Spencer's procedure of slices as coded in the program SLOPE/W (Geo-Slope, Int., 1992). In Spencer's procedure, all forces are assumed to have the same inclination, and all requirements for static equilibrium are satisfied. The trial and error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied. In addition, the slope stability results of some cross sections computed by SLOPE/W also were evaluated using a simplified chart method. The failure surfaces identified by SLOPE/W using Spencer's procedure are circular. For Case II, we used UTEXAS3, which is also a limit equilibrium computer program based on Spencer's procedure, to evaluate the factor of safety against sudden drawdown loading. This computer program is coded with the 3-stage computation that is appropriate for the sudden drawdown loading condition.

4.2.1 Pore Pressures

The SEEP/W program (Geo-Slope International, 1992) was used to estimate the phreatic surface in the levee for the critical flood stage under the steady seepage condition based

on the design water levels provided in the plans prepared by CH2MHILL. The SEEP/W program and the seepage analyses are described in further detail in Section 7 "Seepage Evaluation" of this report. For a given cross section and design condition, the wetted face (phreatic surface) was estimated. The phreatic surface generated by SEEP/W was used to calculate the pore pressures within the levee for Cases II and III.

The phreatic surface at the steady seepage case represents the worst-case highest groundwater condition. Based on the design hydrograph in Figure 3, we developed a time rate of change of the river level (at a given cross section) in order to assess the advancement of the wetted face into the levee during high flow stages. The hydrograph was discussed previously, in Section 1.1 "Project Description". A transient seepage analysis was performed, which demonstrated that the wetted face into the levee never reached the height of the steady seepage condition; therefore the assumption of the piezometric surface is judged to be conservative.

4.2.2 Static Loading Conditions

The static loading conditions for slope stability analysis prescribed by the United States Army Corps of Engineers (Corps) are listed below.

- End of Construction
- Sudden Drawdown
- Steady Seepage from Full Flood Stage

A discussion of the conditions for analysis and their respective strength parameters is found in the following sections.

End of Construction

The end of construction case represents the condition immediately following construction when excess pore pressures have been induced by the construction, and not enough time has passed for the pressures to dissipate. For this reason, undrained conditions exist for an impervious embankment and foundation soils. Strength results from laboratory unconsolidated-undrained tests were used for the fine grained soils and the effective friction angle was used for the granular soils, as they are assumed to be pervious and free draining.

Sudden Drawdown

The sudden drawdown case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream (riverside) embankment portion and then drops faster than the soil can drain. This causes the development of excess pore pressures that may result in the riverside slope becoming unstable.

As discussed above, a three-stage computation was used to estimate the strength of the cohesive soils and effective stresses were used to estimate granular soils. The three-stage computations include: first stage use of effective stresses, second stage use of undrained shear strength and total stresses, and third stage use of drained strengths or undrained

strengths depending on which strength is lower. These strengths are in accordance with the Corps Manual.

Steady Seepage From Full Flood Stage

The steady seepage case represents the condition where the water remains at or near full flood stage so that the embankment becomes fully saturated and a condition of steady seepage occurs. The design shear strength should be based on the effective stresses; this should correspond to the residual strengths where previous shear deformation or sliding has occurred.

For this analysis, the effective stress envelope is based on consolidated undrained test results. Recognizing the non-linear nature of the shear strength envelope for higher effective stresses, we have modified the strength using a composite envelope. Drained strengths were used until they became greater than the undrained strengths. At this point and beyond, an envelope midway between the drained and undrained strengths was used. The water levels used are at the design high levels as specified on the plans prepared by CH2MHILL.

4.2.3 Earthquake Loading Conditions

The earthquake case represents the condition when the levee is subjected to earthquake motion. As discussed above, we performed a 2-step evaluation for the earthquake loading. In the first step, we selected a seismic coefficient equal to one half the peak horizontal ground acceleration as outlined in the 1997 Guidelines for Evaluating and Mitigating Seismic Hazards in California. The peak ground acceleration was determined using a deterministic approach where no consideration is given to the probability of an earthquake occurring. The maximum earthquake magnitudes adopted by the Working Group on Northern California Earthquake Potential (1996) were used for each of the seismic sources. Empirical attenuation relationships were used to calculate the median horizontal ground accelerations as discussed previously in Section 2.3 "Ground Motions". Accelerations were determined for four segments along the reach of the Lower Guadalupe River, as presented in Table 3a through 3d. For the portion of the alignment analyzed in this study (Task 2.4.5), the design seismic coefficient is 0.55g for Cross Sections 14 through 30. The strength parameters used for the earthquake condition were the same as for the end of construction. In the second step, we estimated the permanent deformation at cross-sections with a safety factor less than 1.1 under pseudo-static loading analyzed in the first step.

4.3 LEVEE STABILITY RESULTS FOR 2:1 SLOPES

The minimum (critical) factors of safety computed using the various methods of analysis are summarized in the table below.

	End of Construction (riverside)	End of Constructio n (landside)	Sudden Drawdown (riverside)	Steady Seepage (riverside)	Steady Seepage (landside)	Pseudo-Static (riverside)	Pseudo-Static (landside)

Cross Section No.	Min F.S. = 1.3 Required	Min F.S. = 1.3 Required	Min F.S. = 1.0 Required	Min F.S. = 1.4 Required	Min F.S. = 1.4 Required	Min F.S. = 1.0 Required	Min F.S. = 1.0 Required
14	3.0	2.1	2.0	1.8	1.5	1.2	1.0
15	2.8	3.1	1.8	1.4	1.5	1.2	1.3
16	2.6	2.2	1.8	1.4	1.4	1.2	1.1
17	2.4	2.3	1.7	2.8	1.6	1.0	1.1
18	2.1	2.4	1.9	4.0	2.2	1.0	1.0
19	2.7	2.3	1.4	2.5	1.9	0.9	1.2
20	2.0	2.7	1.3	2.0	1.8	0.9	1.1
21	2.0	3.4	1.1	1.4	1.8	1.0	1.6
22	2.1	3.7	1.5	1.4	1.8	1.0	1.7
23	3.2	1.7	1.5	2.5	2.2	0.7	1.4
24	2.0	2.9	1.3	2.5	2.5	0.8	1.3
25	2.2	6.6	1.0	1.4	1.6	1.0	2.8
26	2.5	5.5	1.4	1.7	1.7	1.5	2.3
27	3.0	Level	1.4	2.5	Level	1.3	Level
28	2.4	Level	1.2	1.4	Level	1.0	1.6
29	2.3	Level	2.0	3.5	Level	0.9	Level
30	2.6	Level	1.2	1.4	Level	1.3	Level

The approximate locations of the critical slip surfaces are shown on Figures 5 through 21.

For a pseudo-static safety factor less than 1.1, we estimated the permanent deformation that could occur under the maximum credible earthquakes. The following table presents a summary of our estimates:

Cross Section No.	Yield Acceleration (g)	Estimated Average Total Deformation (cm)	Estimated Average Vertical Deformation (cm)
14	0.22	22	15
16	0.24	16	11
17	0.25	14	10
18	0.22	16	11
19	0.21	20	14
20	0.22	18	13
21	0.20	20	14
22	0.33	4	3
23	0.15	45	32
24	0.33	4	3
25	0.20	18	13
28	0.32	5	4
29	0.20	14	10

The table above presents results in terms of total deformation (Δ_{total}) along the potential sliding surface and estimated vertical displacement ($\Delta_{vertical}$). For the potential sliding surface through or near the crest of the embankment, $\Delta_{vertical} = 0.7\ \Delta_{total}$ was estimated

based on the slope of the potential failure surface at the crest. As shown, we estimate vertical deformation in the range of 3 to 15 cm (1¼ inch to 6 inch) for Sections 14, 16 through 25, 28, and 29, except at Section 23 where movements are estimated to be 32 cm (12½ inches). At all locations we anticipate that shaking from a major earthquake will result in some cracking of the levee, requiring regrading of the levee.

4.4 LEVEE STABILITY RESULTS FOR 1.5:1 LANDSIDE SLOPES

The minimum (critical) factors of safety computed using the various methods of analysis are summarized in the table below.

Cross Section	Station	End of Construction (landside)	Steady Seepage (landside)	Maximum Steady Seepage (from transient analysis, landside)	Steady Seepage, ϕ = 36° (landside)	Maximum Seepage, ϕ = 36° (from transient analysis, landside)	Earthquake (landside)
		Min F.S. = 1.3 Required	Min F.S. = 1.4 Required	Min F.S. = 1.4 Required	Min F.S. = 1.4 Required	Min F.S. = 1.4 Required	Min F.S. = 1.0 Required
39	E 8+580	3.0	1.1	1.3	1.2	1.4	1.5**
40	E 8+790	3.7	1.3	-	1.4	-	1.7
41	E 10+200	2.5	1.8	-	-	-	1.2**
42	W 10+200	2.9	1.3	-	1.4	-	1.4**
43	E 11+010	3.9	1.3	-	1.5	-	1.6**
44	W 11+010	4.2	1.3	-	1.5	-	1.6
46	E 11+610	3.8	2.4	-	-	-	1.4**
47	W 11+610	3.9	2.3	-	-	-	1.9**
50	E 12+120	6.6	1.2	-	1.4	-	3.0
51	W 12+120	7.3	1.3	-	1.5	-	3.3
52	E 12+300	7.7	1.3	-	1.5	-	3.5**
53	W 12+300	7.3	1.4	-	-	-	3.1
54	E 12+900	5.4	1.3	-	1.4	-	2.3**
55	W 12+900	7.6	1.4	-	-	-	2.7

** Factor of safety computed using Bishop's method (end of construction slip surface assumed).

The approximate locations of the critical slip surfaces are shown on Figures 47 through 60.

5.1 BRIDGES SELECTED FOR ANALYSIS

As part of our scope of work for this project, we analyzed the stability of the slopes at the specified bridge locations. Table 1 presents a list of the bridges crossing the entire project alignment, their approximate station, the proposed changes made to the slope, and comments regarding the type of geotechnical analysis performed. Global stability analyses were performed for the East and West Levees at the Tasman Drive bridge abutments where a cut slope is proposed. The changes to the slope were inferred from the plans prepared by CH2MHILL, dated August 2001, the updated drawings prepared by CH2MHILL from the workshop on October 30, 2001 (Appendix C) and the April 2002 90% Submittal Plans.

5.2 STABILITY ANALYSIS OF SLOPE

5.2.1 General

The global slope stability was performed for the inboard slope of the East and West Levees at Tasman Drive for the following design conditions outlined by the Corps for riverside slopes:

- End of Construction
- Sudden Drawdown
- Critical Flood Stage
- Earthquake (Pseudo-static)

These conditions are described in detail in Section 4 "Levee Slope Stability Evaluation". The changes to the bridge abutments at Tasman Drive were made only on the inboard slopes; therefore, analyses were only performed for the conditions impacting inboard slopes. The peak horizontal ground acceleration at this location is 0.55 g.

5.2.2 Tasman Drive East Levee

The geotechnical parameters used for the analysis are summarized below. The limits of the channel sediments were defined as previously described in Section 4 "Levee Slope Stability Evaluation."

Elevation (feet)	Material	γ_t (pcf)	C' (psf)	ϕ' (degrees)	C (psf)
26 to 0	Lean Clay Fill	132	450	24	1750
0 to –24	Lean Clay	118	35	30	1100
-24 to –30	Silty Sand	135	0	33	--
8 to –24 (within channel)	Lean Clay (Channel Sediments)	118	35	30	500

5.2.3 Tasman Drive West Levee

The geotechnical parameters used for the analysis are summarized below. The limits of the channel sediments were defined as previously described in Section 4 "Levee Slope Stability Evaluation".

Elevation (feet)	Material	γ_t (pcf)	C' (psf)	ϕ' (degrees)	C (psf)
26 to 0	Lean Clay Fill	130	450	24	1850
0 to –24	Lean Clay	118	35	30	1300
-24 to –30	Silty Sand	135	0	33	--
6.5 to –24 (within channel)	Lean Clay (Channel Sediments)	118	35	30	500

5.3 GLOBAL STABILITY RESULTS

The minimum (critical) factors of safety computed for the various cases are summarized in the table below.

		Factor of Safety		
Case	Design Condition	Minimum Required	Tasman Drive East Levee	Tasman Drive West Levee
I	End of Construction	1.3	2.4	2.5
II	Sudden Drawdown	1.0	1.6	1.5
III	Critical Flood Stage	1.4	2.2	2.0
VI	Earthquake	1.0	1.0	1.1

The factors of safety computed for the global stability of the proposed bridge slope at Tasman Drive exceed the minimum required safety factor prescribed by the Corps.

6.1 SUBSIDENCE

From 1916 to 1966 in the San Jose area of Santa Clara Valley, the artesian head declined from 55 to 73 meters (180 to 240 feet) in response to groundwater withdrawal. As a direct result of the artesian head decline, the land surface subsided as much as 3.9 meters (12.7 feet) in San Jose (Poland and Ireland, 1985). The recovery of artesian head since 1967 has been substantial. In downtown San Jose, the artesian head recovered 21.3 to 30.5 meters (70 to 100 feet) in the 8 years to 1975. The subsidence appears to have been stopped by raising the artesian head in the aquifers. From 1934 to 1960, maximum subsidence of 1.5 meters (5 feet) occurred in San Jose and Sunnyvale. From 1960 to 1967, maximum subsidence of 1.1 meters (3-1/2 feet) occurred in San Jose and Santa Clara; the average rate of subsidence was 152 mm (1/2 foot) per year, the most rapid rate of subsidence in any 7-year period. Based on the land subsidence contour map prepared by United States Geological Survey (Poland and Ireland, 1967), the segment of Guadalupe River within the scope of study experienced subsidence ranging from 1.8 to 2.4 meters (6 to 8 feet) between the period of 1934 and 1967. Since 1967, subsidence has been significantly reduced. In San Jose Wells 7S/1E-16C6 and C11, an extensometer recorded compression decreasing from 305 mm (1 foot) in 1961 and 1962 to 76 mm (1/4 foot) in 1967 and to 3 mm (1/100) foot in 1973. Based on the referenced contour map, a maximum drop of 1.1 meters (3½ feet) in elevation of the levees since their improvement construction in 1960 probably can be attributed to land subsidence.

6.2 LEVEE SETTLEMENT

6.2.1 Inferred Settlement

Based on review of the (1) as-built drawings for the 1960 and 1963 improvement construction of the levees and (2) those for the 1983 to 1985 raising and widening of the Guadalupe River channel, we have estimated the levee settlement between Station 7+008 and Station 14+500. Estimates of the settlement are inferred because they are primarily based on the change in elevation of the crest of the levees. Our review indicates that the lowering of crest elevations occurred mostly in areas north of Station 12+000 (Montague Expressway). The levee portions south of Station 12+000 appear to have experienced little or no change in crest elevation. We estimate that the portions of levees north of Station 12+000 settled due to consolidation approximately 229 to 762 mm (average of 381 mm) or ¾ to 2½ feet (average of 1¼ feet) for the West Levee and 396 to 1189 mm (average of 933 mm) or 1¼ to 4 feet (average of 3 feet) for the East Levee for the period of 1960 to 1983. These inferred consolidation settlements deduct the groundwater withdrawal related land subsidence settlement estimated from 1960 to 1967. North of Station 12 +000, we estimated the subsidence settlement to be 457 to 762 mm (1½ to 2½ feet) during this 7-year period.

6.2.2 Predicted Settlement

These estimates have been compared to settlement calculations based on consolidation test results from a 1960 study and our current investigation (2001). It is assumed that the

height of the fill at each stage of construction was added instantaneously. More specifically, the construction stages are presented in the following table.

Cross Section Number	Station	Levee	Fill Thickness (feet)**			
			1963	1983	1995	2002
14	8+280	West	9.5	8.0	2.5	-
15	8+580	East	9.0	6.5	2.5	2.0
16	8+640	West	9.0	7.0	2.5	2.0
17*	9+180	West	11.0	6.0	2.5	1.5
18	9+500	West	7.0	1.5	3.0	1.5
19	9+780	East	6.0	5.5	3.0	1.0
20	10+080	East	6.0	5.5	2.5	1.0
21*	10+470	West	5.0	5.0	3.0	0.5
22*	10+740	East	4.5	5.0	2.5	1.5
23*	11+250	East	7.0	4.5	2.5	2.0
24	11+760	West	3.5	4.0	3.0	2.0
25*	12+120	East	6.0	1.0	1.5	1.0
26	12+810	East	6.0	0.5	1.0	1.0
27	13+860	West	6.0	2.5	1.5	0.5
28	14+370	East	6.5	0	1.0	0
29*	15+176	East	6.0	0	0	0
30	17+370	West	0	0	0	0

* Through channel of old meander.
** Thickness of all fill rounded up to the nearest 0.5 foot.

The locations of the cross sections are shown in plan view in Figure 2, Site and Field Exploration Location Plan. These cross sections are the same cross sections used in the levee slope stability analysis described in Section 4 "Levee Slope Stability Evaluation". The time rate of settlement for the seventeen (17) cross sections are presented graphically on Figures 47 to 63 for the period from 1963 to 2010. The following table summarizes the estimated consolidation settlements from 1963 to 2001.

Cross Section	Estimated Consolidation Settlement					
	From 1963 to 1983		From 1963 to 1995		From 1963 to 2001	
	(mm)	(inches)	(mm)	(inches)	(mm)	(inches)
14	433	17	648	25 ½	698	27 ½
15	403	15 ¾	574	22 ½	622	24 ½
16	422	16 ½	623	24 ½	677	26 ¾
17	427	16 ¾	541	21 ¼	575	22 ¾

Cross Section	Estimated Consolidation Settlement					
	From 1963 to 1983		From 1963 to 1995		From 1963 to 2001	
	(mm)	(inches)	(mm)	(inches)	(mm)	(inches)
18	390	15 ¼	472	18 ½	573	22 ½
19	216	8 ½	349	13 ¾	404	16
20	305	12	524	20 ¾	603	23 ¾
21	205	8	343	13 ½	399	15 ¾
22	233	9 ¼	422	16 ¾	493	19 ½
23	190	7 ½	296	11 ¾	344	13 ½
24	182	7 ¼	395	15 ½	510	20
25	305	12	342	13 ½	388	15 ¼
26	336	13 ¼	373	14 ¾	411	16 ¼
27	341	13 ½	444	17 ½	492	19 ½
28	246	9 ¾	246	9 ¾	281	11
29	260	10 ¼	260	10 ¼	260	10 ¼
30	0	0	0	0	0	0

Cross Sections 14 through 24 are located north of Station 12+000 (near Montague Expressway). More specifically, Cross Sections 14, 16 through 18, 21 and 24 are positioned on the West Levee, whereas Cross Sections 15, 19, 20, 22 and 23 are positioned on the East Levee. The values shown in the above table are consistent with those estimated (inferred) from the As-built drawings. For example, the range of estimated consolidation settlement from the above table for the West Levee for the period 1963 to 1983 range from 182 (Cross Section 24) to 433 mm (Cross Section 14). As described in Section 6.2.1 "Inferred Settlement" the average inferred consolidation settlement for the West Levee was 381 mm. For the East Levee, the range of estimated consolidation settlement from the above table for the period 1960 to 1983 ranged from 190 mm (Cross Section 23) to 403 mm (Cross Section 15). As described in Section 6.2.1 "Inferred Settlement" the range of inferred consolidation settlement for the East Levee was 396 to 1,189 mm. Although the calculated consolidation settlement appears to be closer to the lower range of the inferred consolidation settlement, we judge that the order of magnitude are reasonably comparable considering the crude assumption of the subsidence settlement based on regional data. Therefore, we have concluded the approach and methodology used in our settlement analysis are reasonable for both levees north of Montague Expressway.

Cross Sections 25 through 30 are located south of Station 12+000 (near Montague Expressway). More specifically, Cross Sections 27 and 30 are positioned on the West Levee, whereas Sections 25, 26, 28 and 29 are positioned on the East Levee. Estimated consolidation settlement on the West Levee at Cross Section 30 is nil, which is consistent with the "Inferred Settlement". The range of estimated consolidation settlement on the East Levee from the above table for the period 1960 to 1983 ranged from 246 (Cross

Section 28) to 336 mm (Cross Section 26). As described in Section 6.2.1 "Inferred Settlement" the levee portions south of Station 12+000 appear to have experienced little or no change in crest elevations. We have no rational explanation at the East Levee and at West Levee Cross Section 27 for the discrepancy between estimated consolidation settlement and inferred settlement between 1960 to 1983.

6.3 FUTURE SETTLEMENT

Between Route 237 (Station 8+000) and the Airport Island Bridge (Station 14+700), the levee will be raised a maximum of 1.5 meters (4.9 feet) and the raise decreases in magnitude in a southerly direction. South of the Airport Island Bridge (Station 14+700), the West Levee crest will be unchanged, except for several short segments where maximum rise height is 0.4 to 0.9 meter (1.3 to 3.0 feet).

Most of the levees at the seventeen (17) cross sections will undergo settlement in the future from ongoing consolidation of the foundation clay soil due to fills placed in 1963, 1983, 1995 and presumably in 2002. Consequently, we have estimated the following settlement due to long term consolidation.

Cross Section	Estimated Future Settlement	
	(mm)	(inches)
14	< 8	< ¼
15	31	1 ¼
16	34	1 ¼
17	17	¾
18	46	1 ¾
19	16	¾
20	27	1
21	8	¼
22	36	1 ½
23	33	1 ¼
24	62	2 ½
25	27	1
26	35	1 ¼
27	16	½
28	< 8	< ¼
29	0	0
30	0	0

South of Station 14+700 estimated settlement at Cross Sections 29 and 30 is nil since these areas were not filled and will not be filled. The future settlement north of Station 14+700 is estimated to be on the order of 8 to 62 mm (¼ to 2½ inches).

Upstream of Route 237, some fill will be added behind the five (5) new retaining walls and behind eleven (11) new floodwalls. Additional future settlement is anticipated where

new fill is to be placed behind these walls; the estimated magnitude and rate of settlement of those new fills at the new walls are presented later in Section 9.0 "Floodwall, Retaining Wall, and MSE Wall Design Parameters."

Section 7

7.1 METHOD OF ANALYSIS

The flood levels used in the analysis of each of the selected sections were provided by CH2MHILL on the plans dated August 2001. The flood levels are summarized as follows:

Cross Section Number	Station (meters)	100-year Flood Elevation (meters, NAVD)*	(meters, NGVD)**	(feet, NGVD)**
14	W 8+280	6.00	5.2	17.0
15	E 8+580	6.10	5.3	17.3
16	W 8+640	6.10	5.3	17.3
17	W 9+180	6.28	5.5	17.9
18	W 9+600	6.71	5.9	19.3
19	E 9+780	6.71	5.9	19.3
20	E 10+080	6.89	6.1	19.9
21	W 10+470	7.10	6.3	20.6
22	E 10+740	7.19	6.4	20.9
23	E 11+250	7.62	6.8	22.3
24	W 11+790	7.92	7.1	23.3
25	E 12+120	8.29	7.5	24.5
26	E 12+810	8.53	7.7	25.3
27	W 13+890	9.75	8.9	29.3
28	E 14+370	10.30	9.5	31.1
29	E 15+176	12.10	11.3	37.0
30	W 17+370	16.70	15.9	52.3

* From August 2001 Drawings, CH2MHILL
** Converted to NGVD Datum

Seepage through the levee during maximum flood was analyzed using a steady state analysis. A finite element program, SEEP/W [developed by Geo-Slope International (1992)], was used to perform calculations. SEEP/W is a computer program of two-dimensional code with isoparametric and higher order finite elements capable of solving saturated and unsaturated flow conditions using steady state and transient analysis. The program can represent multiple soil types with anisotropic hydraulic conductivity; it can also represent prescribed constant and transient boundary conditions.

The two boundary conditions established for the steady state analysis are constant head and no flow conditions. The nodes beneath the water on the inboard side of the levee used the boundary condition of head equal to the height of the design water level. A boundary condition of no flow was established across the bottom of the finite element mesh. The nodes on the upstream portion of the levee were assigned a flux-type review flow boundary condition. In this case, after the heads are computed for all nodes, the review nodes are modified if any one of the review nodes has a computed head greater than the y-coordinate of the node (that is, if the porewater pressure is greater than zero). Using the maximum pressure review option, the review node with the maximum positive pressure head is identified by SEEP/W. The head at this node is then set to the

y-coordinate for the next iteration. At the outboard boundary of the mesh, a constant head boundary was established. The constant head value was inferred from water level observed in historical borings in the vicinity.

7.2 CROSS SECTIONS AND RESULTS OF ANALYSIS

In general the levees are constructed of well compacted lean clays or lean to fat clays. Furthermore, most of the native soils directly underlying the levee fills are lean to fat clays. Since the clayey soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be very low. To confirm these opinions, we performed a steady-state seepage analysis on the 17 critical cross sections. These cross sections correspond to the cross sections listed and described in Section 4.1 "Cross Sections Selected for Analysis."

The coefficients of hydraulic conductivity (permeability) for three types of soil (lean clay, silty sand and sand) were selected from the following laboratory and published data. They included laboratory permeability test results of relatively undisturbed and remolded samples, broad empirical relationships between soil types and permeability (Cedergren, 1967) as shown in Figure 64, and correlation relationships based on grain size distribution as described in the Corps' EM-1110-2-1913 (COE, 1978) and the NAVAC DM-7.1 (1982). Permeability tests were performed for the 2000 study on three relatively undisturbed soil samples as reported in Appendix G of that report. The coefficient of hydraulic conductivity (permeability), k, determined from these samples are as follows:

Sample Number	Depth (feet)	Soil Type	Location	Dry Density (pcf)	k (cm/second)
EB 03-16-4	30 – 31.5	Lean Clay	Foundation	101.5	3.3×10^{-7}
EB 11-9-4	14.5 – 16	Silty Sand	Foundation	90.3	5.1×10^{-4}
WB 10-11-4	26 – 27.5	Silty Sand	Foundation	125.2	1.2×10^{-4}

The values assigned to the corresponding soil types are presented in the following table. The ratio of anisotropy of the horizontal to vertical conductivity was assumed to be 4 for all soil types.

Soil Type	Coefficient of Hydraulic Conductivity (k_h)		Selected Values	
	cm/sec	ft / hr	cm/sec	ft / hr
Sand	10^{-2} to 10^{-4}	1.2 to 0.012	10^{-2}	1.2
Silty Sand, Clayey Sand	5×10^{-3} to 5×10^{-4}	0.7 to 0.07	6×10^{-3}	0.7
Lean to Fat Clay	10^{-5} to 10^{-6}	1.2×10^{-3} to 1.2×10^{-4}	10^{-5}	1.2×10^{-3}

The approximate steady-state phreatic surface line is presented on Figures 5 through 21 for Cross Sections 14 through 30, respectively. A total of seven representative or critical cross sections were selected for detailed presentation. The SEEP/W output for the steady state condition at the seven cross sections is presented in Figures 78 through 84 for Cross Sections 15, 16, 19, 22, 23, 25 and 28. These figures also present the estimated flow

quantities in cubic feet per hour per linear foot for a steady state seepage condition with river levels at 100-year elevations, as summarized below.

Figure Number	Cross Section Number	Levee	Station	Estimated Seepage Quantity: Cubic Feet per Hour per Linear Foot	Estimated Seepage Quantity: Gallons per Minute per Linear Foot	Comment
	14	West	8+280	6.016E-04	7.50E-05	
78, 85	15	East	8+580	7.770E-04	9.69E-05	
79, 86	16	West	8+640	7.796E-04	9.72E-05	
	17	West	9+180	8.571E-04	1.07E-04	
	18	West	9+600	7.361E-05	9.18E-06	
80, 87	19	East	9+780	4.844E-04	6.04E-05	Sand Layer
	20	East	10+080	1.577E-04	1.97E-05	
	21	West	10+470	5.600E-05	6.98E-06	
81, 88	22	East	10+740	7.682E-05	9.58E-06	
82, 89	23	East	11+250	6.332E-04	7.89E-05	Sand Layer
	24	West	11+790	9.376E-04	1.17E-04	
83, 90	25	East	12+120	1.122E-04	1.40E-05	
	26	East	12+810	6.144E-05	7.66E-06	
	27	West	13+890	6.396E-05	7.97E-06	
84, 91	28	East	14+370	7.307E-05	9.11E-06	
	29	East	15+176	4.513E-05	5.63E-06	
	30	West	17+370	NA	NA	

* See discussion below

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee at the above 17 sections is expected to be less than 1/10 gallon per minute (gpm). In our opinion these seepage quantities are very small. However, it should be emphasized that the quantity of estimated seepage refers to the underground water flow through a theoretical cross-section at the toe, i.e. no seepage exiting the toe of the levee.

The above discussion is for a steady state condition such that seepage through the levee achieves equilibrium during the 100-year flood elevation. To simulate the change in river level during the flood event, we performed a transient analysis for the seven (7) selected sections based on the 100-year flood hydrograph (Figure 3). The results of the transient analysis are presented in Figure 85 through Figure 86. These figures present the phreatic surface through the levee at different times. For example, in Figure 85, the initial groundwater level is shown as line 0; subsequent phreatic surfaces at times of 12 hours (time step 6), 24 hours (time step 12), 48 hours (time step 24), 72 hours (time step 36) and 84 hours (time step 42). As shown on Figure 85, the highest phreatic surface is at 48 hours; this surface is lower than the steady state phreatic surface shown in Figure 78. This observation is similar to the other cross sections. By comparison, we conclude that the estimated seepage quantities will be much less than the ¼ gpm estimated previously.

All sections indicate relatively thick layers of lean clay soils where seepage would likely exit at the outboard toe of the levee except at Cross Section 19. Due to the cohesiveness and the low permeability of these soils, piping is unlikely.

For Cross Section 19, the lean clay soil cover at the outboard toe is about 0.6 meter (2 feet). We estimate the exit upward gradient to be about 0.3, which is less than Corps' requirement of 0.5. Therefore, we expect the potential for hydraulic failure and development of sand boils at the outboard toe to be low.

8.1 SITE AND SOIL CONDITIONS

Along the planned alignment of the setback levee, the existing ground is relatively level. It is covered by light vegetation consisting of brush and weeds. There is some evidence of fill materials as it appears to have been used as a staging area for fill and borrow materials for nearby construction sites. Sources of these fill materials are unknown.

The subsurface condition was investigated by drilling four exploratory borings (WB-26 to WB-29). In addition, we have reviewed available data from nearby exploration. These exploratory data revealed a fill material ranging from 2 to 2.6 meters (6 to 9 feet) in depth underlain by alluvial deposits of lean clays with gradational sands and silts interbeds. The fill material consists of stiff to hard lean and fat clays. The underlying native lean clays vary from medium to stiff in consistency with gradational sands and silt interbeds.

8.2 CROSS SECTIONS FOR ANALYSIS

Two representative cross-sections (Cross Sections 33 and 34) were selected for geotechnical analyses including slope stability, settlement and seepage. The approximate locations of these cross sections are presented on Figure 2. These cross sections were selected on the basis of similar subsurface conditions and highest slope height of the finished levee section. Therefore, we judge the results of the geotechnical analysis to represent reasonably the most critical cases.

8.3 RESULTS OF GEOTECHNICAL ANALYSIS

8.3.1 Slope Stability

Employing the same method of analysis and selection of shear strength parameters discussed in Section 4 "Levee Slope Stability Evaluation", we have analyzed the setback levee for four design conditions. Below is a summary of the results:

Cross-Section No.	End of Construction (riverside)	End of Construction (landside)	Sudden Drawdown (riverside)	Critical Flood (riverside)	Steady Seepage (landside)	Earthquake Pseudo Static (riverside)	Earthquake Pseudo Static (landside)
	Min F.S. = 1.3 Required	Min F.S. = 1.3 Required	Min F.S. = 1.0 Required	Min F.S. = 1.4 Required	Min F.S. = 1.4 Required	Min F.S. = 1.0 Required	Min F.S. = 1.0 Required
33	3.9	8.4	1.7	2.7	2.0	1.8	2.6
34	3.6	13.6	1.6	2.5	3.9	1.6	3.3

The results of the above cases of analysis are presented graphically in Figures 92 and 93 for Cross Sections 33 and 34, respectively. The selected design parameters for the analysis are based on index property plots shown on Figures 94 and 95.

8.3.2 Settlement

Using the same settlement calculation method and assumptions discussed in Section 6 "Settlement Evaluation", we estimated settlement due to construction of the new levee fill along the planned setback alignment. The table below summarizes the results of the settlement analysis:

Cross-Section No.	Maximum Fill Height	Estimated Ultimate Consolidation		Estimated 90% Consolidation		Time to Ultimate Settlement	Time to 90% Consolidation
	(ft)	(in)	(mm)	(in)	(mm)	(months)	(months)
33	6.2	2.1	53	1.9	48	5	2
34	3.5	1.9	48	1.7	43	12	7

Time rates of the estimated settlement are presented in Figures 96 and 97 for Cross Sections 33 and 34, respectively.

8.3.3 Seepage

To estimate the seepage quantity through and under the new levee, as well as the hydraulic gradient at the outboard toe during the 100-year flood event, we have used the same approach and assumptions discussed in Section 7 "Seepage Evaluation". For the levee fill material, we expect as a minimum, it will have a similar permeability as the existing levee fill material. With this assumption and the subgrade condition we encountered in the exploratory borings, we estimate the seepage quantity to be less than 1/10 gallon per minute per 30 meters (100 feet) of the levee section. The exit gradient is expected to be less than 0.1, which is below the Corp's requirement of 0.5. The results of the steady state analysis are presented graphically in Figures 98 and 99. Results of the transient analysis based on the project hydrograph is presented in Figures 100 and 101 for Cross Sections 33 and 34, respectively.

Section 9

9.1 FLOODWALL

9.1.1 Background

As noted in the July 2002 Design Drawings, the levee improvements between Station 8+000 and Station 8+540 include the construction of a reinforced concrete floodwall on both the East Levee and West Levee. The floodwall varies in height (above existing ground surface) from 0.7 to 1.2 meters (2.3 to 3.9 feet). The vertical floodwall will be positioned with the vertical face matching the inboard hinge point of the crest; this will result in access to a full width maintenance road even during the 100-year design flood. In addition, short floodwalls are planned at the four corners adjacent to the bridge crossings at Montague Expressway and Highway 101, as well as an approximately 80-meter long floodwall on the West Levee at the Hetch-Hetchy Pipeline Crossing.

We provided preliminary design parameters for the floodwalls in our April 14, 2000 and October 23, 2001 letters to the District and to CH2MHILL, respectively. A copy of each letter is presented in Appendix D. These parameters are defined by reach as designated by CH2MHILL. As discussed in Section 1.1 "Project Description", Reach F extends from Station Limits 8+000 to 9+200. Since the floodwalls extend from Station 8+000 to Station 8+540, they are located exclusively in Reach F. Therefore the following discussion applies to Reach F.

9.1.2 Corps Requirements

We have reviewed (1) our preliminary design parameters in the two letters listed in Appendix D and (2) the CH2MHILL Map and General Plan drawings dated August, 2001. At that time no specific design system was utilized, other than some Caltrans criteria. Subsequent to issuing our draft December 7, 2001 report, we now understand that Corps criteria will be used. Therefore, we have revised our recommendations consistent with the Corps Manual No. EM 1110-2-1913 (Design and Construction of Levees, April 30, 2000) and Corps Manual No. EM 1110-2-2502 (Engineering and Design, Retaining and Flood Walls, September 29, 1989). According to EM 1110-2-1913, levee enlargements "are constructed either by adding additional earth fill or by constructing a flood-wall, "I"-type or "inverted T"-type, on the crown."

The I floodwall is a vertical wall partially embedded in the levee crown. The stability of such walls depends upon the development of passive resistance from the soil. For stability reasons, I floodwalls rarely exceed 2.13 meters (7 feet) above the ground surface, according to the Corps. One common method of constructing an I floodwall is by combining sheet piling with a concrete cap. The lower part of the wall consists of a row of steel sheet piling that is driven into the levee embankment, and the upper part is a reinforced concrete section capping the steel sheet piling.

An inverted T floodwall is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic-pressures acting against the wall. According to

the Corps, the inverted T floodwall is used to make floodwall levee enlargements when walls higher than 2.13 meters (7 feet) are required.

As described in the 30-*Percent* Design Submittal, CH2MHILL analyzed the following three wall types.

- Cantilever L-type wall on spread footing (refer to Figure A-2 in the 30-Percent Design Submittal);
- Cantilever inverted T-type wall on spread footing (refer to Figure A-3 in the 30-Percent Design Submittal); and
- Cantilever I-type wall on 600-mm-diameter (24 inches) cast-in-drilled-hole (CIDH) pile with a pile cap (refer to Figure A-4 in the 30-Percent Design Submittal).

Based on their analysis, CH2MHILL recommended that the cantilever L-type wall with spread footing be constructed because of its low initial costs and simpler construction methods. However, we understand the shear key has been moved to coincide with the riverside edge of the footing.

9.1.3 Wall Settlement

As described in Section 6.3 "Future Settlement", we have estimated future total settlement of the existing levees in Reaches F, C, B and A to be of order 8 to 62 mm (¼ to 2½ inches). This is the result of ongoing consolidation settlement of the foundation clay layers as a result of the 1963, 1983 and 1995 levee construction and fill placement activities. Based on this total settlement, we estimate maximum differential settlement of the levee and cantilever L-type floodwall to be approximately 12 mm (½ inch) in a distance of 30 m (100 feet).

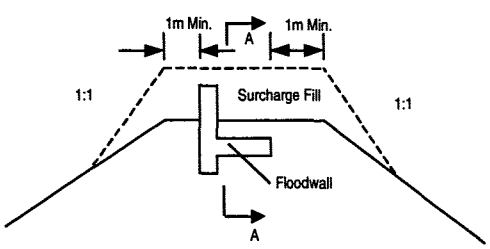
We have also estimated additional settlement at FW4W, FW5E and FW6W due to new fill to be placed behind the floodwall ranging in height from 0.8 to 1.3 meters (2.6 to 4.3 feet). The estimated additional settlement and rates, including the effect of surcharging each area with 1.5 meters (5 feet) of temporary fill, is shown in the following table.

Wall	Station	Cross Section Used in Analysis	Max. New Fill Height (m)	Estimated Time Rate Settlement** (mm)				Ultimate Settlement from New Fill (mm)	90% Consolidation (mm)	Surcharge? ($\delta_{90\%}$ > 25.4 mm) (1 inch)	Time to 90% Consolidation (months)	
				1 month	3 months	6 months	1 year				No Surcharge	w/1.5 m Surcharge
FW4W	W41+449 to W41+843	#14	0.8	20	27	32	34	37	33	yes	9	1
FW5E	E22+292 to E22+730	#15	0.7	6	9	12	16	20	18	yes	18	2
FW6W	W43+341 to W43+430	#14	0.5	15	21	24	26	28	25	yes	9	1
FW7E	E25+827 to E25+855	#23	1.0	15	25	31	33	36	32	yes	12	1
FW8E	E25+888 to E25+923	#23	0.8	16	26	31	34	37	33	yes	12	1
FW9W	W45+038 to W45+061	#24	1.1	41	52	58	62	67	61	yes	12	1

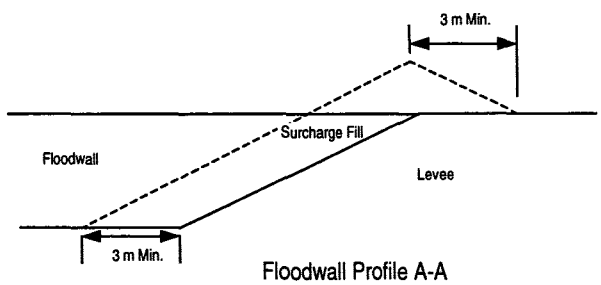
Wall	Station	Cross Section Used in Analysis	Max. New Fill Height (m)	Estimated Time Rate Settlement** (mm)				Ultimate Settlement from New Fill (mm)	90% Consolidation (mm)	Surcharge? ($\delta_{90\%}$ > 25.4 mm) (1 inch)	Time to 90% Consolidation (months)	
				1 month	3 months	6 months	1 year				No Surcharge	w/1.5 m Surcharge
FW10W	W45+094 to W45+217	#24	0.6	24	30	34	37	41	37	yes	12	1
FW9E	E28+576 to E28+580	#28	1.8	35	54	65	71	75	68	yes	12	1
FW11W	W47+730 to W47+732	#27	0.4	7	11	15	18	24	21	no	24+	1

As shown above, the time to achieve 90% consolidation is about 9 to 24 months. If a surcharge height of 1.5 meters (5 feet) is placed for a period of about 2 months at FW5E and about 1 month at the remaining floodwalls, 90% consolidation settlement could be achieved. This would result in post-surcharge settlement of less than 6 mm (¼ inch), assuming the walls are constructed after surcharge is removed. If surcharge is selected, we recommend that settlement plates/devices be installed prior to placement of the fill and settlement be monitored before, during and afterwards. These monitoring results should be submitted to and reviewed by URS.

Based on the estimated settlement, we estimate maximum differential settlement of the levee and cantilever L-type floodwall to be approximately 6 mm (¼ inch) in a distance of 30 m (100 feet). The following sketches illustrate the lateral extent of the surcharge fill we recommend.



Typical Floodwall Cross-Section



Floodwall Profile A-A

If this estimated settlement is acceptable to the Structural Engineer and the District, then spread footing foundations would be appropriate. Otherwise, drilled piers are feasible, but should extend below the compressible foundation clay layers. Although the cantilever L-type floodwall will likely be used, recommendations for both foundation types are presented below.

9.1.4 Driving Side Design Earth Pressure

Consistent with the Corps requirements, a tension crack should be assumed to extend vertically down to the bottom of the shear key on the riverside. Therefore, the floodwalls should be designed to resist an "at-rest" earth pressure equal to water with an equivalent fluid pressure of 1 Mg/m^3 (62.4 pcf). This value is recommended, regardless of foundation type.

9.1.5 Resisting Side Design Earth Pressure

At Rest Earth Pressure

The estimated moist unit weight of the clay soil adjacent to the floodwall and the corresponding submerged unit weight of the same clay soil are shown in Table 8. Assuming that soil will not be excavated adjacent to the wall during its' lifetime, at-rest conditions occur on the resisting side. For this at-rest condition, the coefficients of lateral earth pressure shown on Table 8 are recommended to estimate the pressures against the retaining walls. Use the moist unit weight when groundwater is at the bottom of the footing at the landside edge. When groundwater rises to ground surface above the landside footing edge, use the submerged unit weight to estimate the equivalent earth pressures and then plus the hydrostatic water pressure. The wall resistance to sliding should be evaluated for both loading cases that represent different stages of seepage. The upper 300 mm (1 foot) of embedment should be neglected for design.

We understand floodwalls will be designed with a shear key. Where there is a shear key or the bottom of slab is inclined, the shear resistance through the clay subgrade should be estimated using the cohesion (undrained condition) or friction angle (drained condition),whichever is less, shown in Table 8.

In the event that a spread footing (horizontal base) with no shear key is selected at a few locations, an ultimate coefficient of friction of 0.325 (tangent of 18 degrees for clay to concrete friction angle) should be used between the base of the retaining wall and underlying soil, with the total resistance not to exceed 36 kPa (750 psf). This assumes (1) that the retaining walls are cast neat against undisturbed engineered fill, (2) the bottom of slab is horizontal and (3) there is no shear key.

Passive Pressure

In accordance with the Corps manual for certain loading conditions, resisting-side earth pressure may be increased, but not to exceed one-half the (ultimate) passive pressure using unfactored shear strengths. Again, assuming that soil will not be excavated to the wall during its' lifetime, an ultimate passive resistance of the soil should be estimated using the coefficients of passive earth pressure shown in Table 8. With groundwater is the bottom of the footing at the landside edge, use the moist unit weight of soil. When groundwater rises to ground surface above the landside footing edge, use the submerged unit weight to estimate the passive earth pressure and then add the hydrostatic water pressure. The wall resistance to sliding should be evaluated for both loading cases that represent different stages of seepage. The upper 300 mm (1 foot) of embedment should be neglected for design.

9.1.6 Bearing Capacity

Spread footings, when supported on new or existing engineered fill material consisting of stiff to very stiff clay soils, should be designed using the ultimate bearing capacity shown

in Table 8. For the load cases I1 through I4 (Paragraph 4-4 of EM 1110-2-2502), the calculated toe pressures should be checked against the ultimate bearing capacity such that the factor of safety exceeds those shown on Table 4-2, EM 1110-2-2502.

It is recommended that the bottom of footing be placed a minimum of 0.91 meter (3 feet) below lowest adjacent finished grade.

9.1.7 Foundation Uplift

A spread footing foundation will be temporarily subject to seepage uplift during the 100-year flood event. We recommend the water pressure on the base be estimated by the line-of-creep method presented in the Corps Manual EM 1110-2-2502.

9.1.8 Drilled Piers

We understand there are no published Corps guidelines or manuals for design of drilled piers. Therefore, in the event drilled piers are selected by the Design Team using the Caltrans method (Memo to Designers 22-1, dated July 2001), we recommend the geotechnical design parameters in the following table.

Geotechnical Parameter	Applicability	Recommended Value
Design Unit Skin Friction for Piles	All Floodwalls	24 kPa (500 psf)
Isolation Factor	All Floodwalls	1.0
Ka (active coefficient)	FW5E, FW4W	0.32
Ka (active coefficient)	FW7E, FW8E, FW9E, FW10E FW6W, FW9W, FW10W, FW11W, FW12W	0.30
Kp (passive coefficient) 2H:1V Slope Downward	All Floodwalls	1.7
Kp (passive coefficient) Horizontal	All Floodwalls	3.2

9.1.9 Construction Considerations

The conditions encountered in the borings consist primarily of layers of lean clay and fat clay. Groundwater was encountered at about Elevation 1 to 2 m, which corresponds to the approximate existing channel bottom. The April 2002 drawings also show a mean daily high tide at about Elevation 1.9 to 2.1 m in this area. Therefore, during construction we would expect groundwater as high as Elevation 2.1 or higher depending on rainfall. While the cohesive deposits will help to keep the hole open during construction, some sloughing and caving can be expected if cohesionless deposits are encountered. Therefore, it is recommended that the Contractor have casing on site, should it be needed to minimize the risk of caving in either clay or sand soils. If casing is used, the casing should be withdrawn from the hole slowly as the concrete is being placed; a minimum head of concrete of 0.9 meter (3 feet) should be maintained in the casing at all times. All

CIDH piles should be installed under the direct observation of a representative of the Geotechnical Engineer.

9.2 RETAINING WALL DESIGN

9.2.1 Wall Configurations

Based on the 90% Design Submittal, we understand that eight retaining walls are planned on the inboard toe of the levees, whereas two retaining walls are planned on the outboard toe of the levees. The inboard walls are planned to retain fill primarily to accommodate the ramps accessing the lower maintenance roads from the levee crest, whereas the outboard walls are needed because of limited District right-of-ways. Table 9 summarizes the planned configurations of these walls.

9.2.2 Design Criteria

According to Corp guidelines (EM 1110-2-2502), the walls should be evaluated for the following three loading conditions:

- Case R1, Usual Loading: the backfill is in place to the final elevation with surcharge load applied.
- Case R2, Unusual loading: same as Case R1, except other temporary loads are considered.
- Case R3, Earthquake Loading: same as Case R1 with the addition of earthquake loads.

The guidelines also outline the need for stability evaluation for three failure modes: (1) sliding at the base, (2) overturning of the wall and (3) bearing failure. These failure modes are shown graphically on Figure 4-1 of the Corps manual EM 1110-2-2502.

For non-free draining foundation soils, both effective and undrained strength parameters should be used to evaluate for the loading conditions. For free draining foundation soils, the evaluation should be based on effective strength parameters. These parameters for the corresponding loading conditions are summarized in Table 10.

9.2.3 Lateral Pressures

Soil lateral pressures for static loads (Cases R1 and R2) were estimated for the driving and resisting sides of the wall. For the driving side lateral pressure, an at-rest backfill condition (k_0) was assumed, as recommended by the guidelines (USACE, 1989). This at-rest lateral pressure coefficient was estimated using the Jaky's equation (1944) and then compared with Coulomb active earth pressure theory modified with a Strength Mobilization Factor (SMF) of 2/3; the higher value is selected for design. For the resistant side lateral pressure, a passive soil condition was assumed without a strength modification (SMF = 1.0). Table 10 summarizes the soil parameters and lateral soil coefficients we recommend for the three loading conditions.

Soil lateral pressure for earthquake loads (Case R3) should be estimated using the general wedge earthquake method. This method is outlined in Paragraph 3-26, Sub-paragraph c of the Corp manual (page 3-65 of the EM 1110-2-2502). Based on a discussion between CH2MHILL and the Corp, we understand that these retaining walls are to be evaluated for the Operating Basis Earthquake (OBE) ground motion. Table 10 also presents the seismic coefficients that are the average horizontal and vertical accelerations acting on the seismic wedge. These coefficients are one half the ground accelerations due to the OBE events and are judged to be appropriate for the wedge based on our past project experience.

9.2.4 Bearing Capacity

Spread footings, when supported on existing engineered fill material overlayed by native soft to medium clays or directly on native clays, should be designed using the ultimate bearing capacity shown in Table 10. For the load cases R1 through R3 (Paragraph 4-3 of EM 1110-2-2502), the calculated toe pressures should be checked against the ultimate bearing capacity such that the factor of safety exceeds those shown on Table 4-1, EM 1110-2-2502.

It is recommended that the bottom of footing be placed a minimum of 0.91 meter (3 feet) below lowest adjacent finished grade.

9.3 MSE WALL DESIGN

9.3.1 Wall Configuration

Based on the 90% Design Submittal and subsequent information provided by CH2MHILL (Appendix E), we understand that eight Mechanically Stabilized Embankment (MSE) walls are planned at the outboard toe of the levees. The reason for these MSE walls is the limited District right-of-ways. The planned configuration of these walls was previously described in Section 1.1 "Project Description." Typical MSE wall orientation as well as typical loading conditions are shown on Figure 108 which summarizes the planned configuration of these walls.

9.3.2 Materials

The embankments behind the MSE walls are planned to be constructed either horizontal or at 2 to 1 (horizontal to vertical) slopes. The MSE walls should be constructed with compacted, relatively clean, granular imported material meeting the requirements of Caltrans Standard Specifications Section 19-3 and the following criteria.

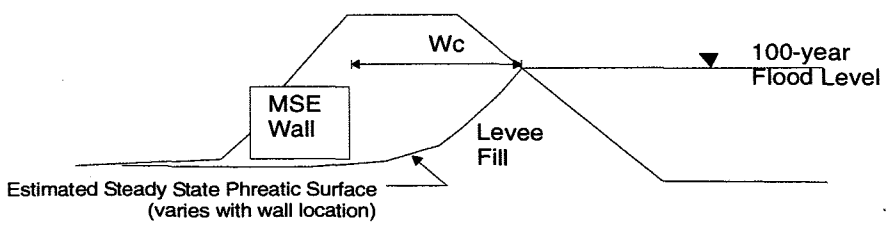
- Particle size ≤150 mm
- Amount passing the 76 mm Sieve, 78 to 100%
- Amount passing the 0.074 mm (No. 200 Sieve) ≤25%
- Plasticity Index < 10
- Angle of Internal Friction, $\phi \geq 34$ degrees
- Sand equivalent ≥ 20

9.3.3 Seepage Considerations

Since an MSE wall typically consists of granular backfill reinforced with metallic strips or wire mesh, the wall unit is expected to have a higher permeability than the adjacent levee fill materials. To evaluate any adverse effect on the seepage control characteristics of the levees, we elected to establish an effective minimum section that a levee should have under the design 100-year flood event and then compare that to the planned MSE wall configurations. This effective minimum section is based on the following guidelines:

- A minimum crown width of 3.05 to 3.66 meters (10 to 12 feet) as outlined in the Corps' Manual for "Design and Construction of Levees", EM 1110-2-1913, Chapter 6.
- A minimum impervious core width of 3.3 meters (10 feet) for general dam/levee safety design.

Based on the above guidelines and our experience, we recommend a practical minimum section of 3.3 meters (10 feet) of impervious clay levee fill be required for these levees under the design 100-year flood event. Please see the following sketch illustrating the measurement of the effective minimum section (Wc).



In addition, we have estimated the seepage paths using SEEP/W finite element seepage analysis for the various levee cross-sections. We have assumed steady state seepage using 100-year flood level. Based on the seepage analysis, we compared the resultant phreatic surface with the proposed configuration of the MSE walls.

Our evaluation indicates that all proposed MSE walls meet the required minimum section Wc of 3.3 meters (10 feet). Our analysis also indicates the phreatic surface will fall below the bottom of some MSE walls ;therefore, it is unlikely that seepage from the river (100-year flood event) will intercept walls MSW3E, MSW4E, MSW2W and MSW4W.

For walls MSW2E, MSW5E, MSW1W and MSW3W, the lower third of these walls may intercept the seepage paths if steady-state seepage is developed. Based on our transient seepage analysis for the design hydrograph (Figure 3), we estimate the wetting front to be well below the steady state phreatic surface; therefore, the seepage analysis for the steady state condition is conservative for the 100-year flood event. Nevertheless, seepage through the levee and then through these four MSE walls should be anticipated in design. We estimate the seepage quantity flowing into the MSE granular backfill will be relatively small (below 1/10 gallon per minute per lineal foot of wall) and the gradient at the exit toe to be less than 0.5. Since these walls consist of granular backfill, we expect

the small quantity of floodwater, if any that might seep through the backfill and eventually exit the walls through the front panel joints would result in little to no buildup of hydrostatic pressure in the walls. We have assumed that an envelope of permeable material will be placed between the granular backfill and the precast concrete wall panels, and geotextile filter fabric will be placed between the permeable material backfill and concrete wall panels.

In addition, we recommend that new fill placed above the MSE walls (mostly where there is a sloping backfill) consist of relatively impervious lean clay. If possible, this compacted clay layer should be at least 1 meter (3.3 feet) thick. The purpose of this clay cap is to minimize rainfall infiltration behind the precast concrete wall panels.

Since these walls meet the minimum levee section required and the potential seepage quantity is low, if any, we expect the MSE wall to have no adverse effect on the seepage control characteristics of these levees.

9.3.4 Driving Side Lateral Pressures

The MSE walls should be designed to resist lateral earth pressures imposed by the adjacent backfill. It is recommended that the walls be designed to resist a lateral pressure based on the following coefficient, K_a.

Retaining Wall Number	Backfill Slope β	Unit Weight of Adjacent Levee Fill (kN/m^3) γ_s	K_a
MSW1W	2H : 1V (26.6 deg)	20.7	0.37
MSW2E	2H : 1V (26.6 deg)	20.7	0.39
MSW2W	2H : 1V (26.6 deg)	20.3	0.41
MSW3E	Horizontal	20.1	0.32
MSW3W	2H : 1V (26.6 deg)	21.7	0.40
MSW4W	Horizontal	21.7	0.32
MSW4E	Horizontal	21.7	0.32
MSW5E	Horizontal	20.4	0.32

Note that for walls with sloping backfill, the pressure diagram behind the reinforced fill section should extend to the top of levee or slope (see Figure 108). Surcharge loads such as vehicular traffic would also apply additional loads to the walls. For design purposes, a surcharge pressure equivalent of 0.6 meter (2 feet) of soil should be applied to account for traffic loads. A surcharge unit weight of 20 kN/m^3 can be assumed for design.

The recommended pressures assume that no hydrostatic pressures will develop behind the MSE walls. The buildup of hydrostatic pressure behind the walls should be prevented by the inclusion of a positive drainage system. It is recommended that such a system consist of a series of permeable material wedges placed behind the precast concrete wall panels.

9.3.5 MSE Stability

The following discussion presents geotechnical parameters for evaluation of the external stability of MSE walls. For internal stability of the wall elements, the Structural Engineer's design should be consistent with the wall manufacturer's published guidelines and/or Caltrans design procedures.

Sliding and Overturning Stability

It is recommended that the sliding and overturning stability of the MSE walls be evaluated during design using the following parameters:

Engineering Property	Granular MSE Backfill	Foundation Soil							
		MSW1W	MSW2E	MSW2W	MSW3E	MSW3W	MSW4W	MSW4E	MSW5E
Friction Angle, ϕ (degrees)	34	0	0	0	0	0	0	0	0
Cohesion, C (kPa)	0	38	48	48	72	98	98	70	48
Total density, Υ (kN/m^3)	19	16	16	16	16	16	16	16	16

Refer to Figure 108 for the loading diagram which presents the sliding forces and overturning moments. For sliding stability, a friction coefficient of 0.48 (ultimate) should be assumed to exist at the base of the wall to resist sliding. The friction coefficient is based on the friction resistance between the granular backfill and reinforcement, i.e. tangent 25.5 degrees where 0.75x34 degrees =25.5 degrees.

We recommend that the walls be designed to achieve minimum stability safety factors of 1.5 for static and 1.1 for seismic conditions for sliding mode of failure. For overturning stability, the minimum factor of safety should exceed 2.

Global Stability

Global stability is defined as the evaluation of a deep seated failure, i.e. below and outside the MSE wall elements. The global stability of the MSE walls was evaluated using SLOPE/W (Geo-Slope, Int., 1992). In Spencer's procedure, all forces are assumed to have the same inclination, and all requirements for static equilibrium are satisfied. The trial and error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied.

The global stability of the eight retaining walls was analyzed for the end of construction condition. The end of construction case represents the condition immediately following construction when excess pore pressures have been induced by the construction, and not enough time has passed for them to dissipate. For this reason, undrained conditions exist

for impervious clay soils, whereas drained strengths were used for granular soils. Strength results from laboratory unconfined compressive strength tests were used for the fine grained soils and the estimated friction angle was used for the granular soils. In our engineering opinion, the end of construction represents the most critical loading condition.

The selection of cross-sections for the global stability evaluation of the MSE walls is consistent with the selection of cross-sections and generalization of subsurface conditions for the levee reaches. As discussed in Section 4, we had subdivided the levee into 23 sections in order to evaluate the stability of the 17 km of levees (combining East and West Levees) systematically. Within each levee section, the subsurface conditions are assumed to be the same. Then, based on the geometry of the levee and corresponding ground condition, we selected the critical cross-section for analysis. For the MSE walls, we first determined what levee section would be appropriate for the MSE wall based on locations. Then, we selected the closest cross-section to that wall as the basis for global stability and seepage analyses. However, the Corp felt that more subsurface data are required at the actual wall locations. Therefore, they recommended that provisions should be made to perform cone penetration tests (CPT) during construction as field confirmation of the assumed conditions. For confirmation purposes, we believe that CPTs at a spacing of 50 m along walls would be reasonable.

The minimum required factor of safety for global stability is 1.3.. The minimum (critical) factors of safety computed are summarized in the table below.

Retaining Wall No.	Minimum Calculated Factor of Safety
MSW1W	2.8
MSW2E	3.4
MSW2W	3.5
MSW3E	8
MSW3W	6.8
MSW4W	11
MSW4E	5.5
MSW5E	5.6

The calculated factors of safety all exceed the minimum required value of 1.3.

9.3.6 Bearing Capacity

MSE walls should be designed for the following ultimate bearing pressure values:

Retaining Wall No.	Ultimate Bearing Pressure (kPa)
MSW1W	215
MSW2E	275
MSW2W	275
MSW3E	407

MSW3W	443
MSW4W	419
MSW4E	335
MSW5E	275

We recommend that the walls be designed to achieve a bearing pressure less than one half the ultimate bearing capacity.

9.3.7 Wall Settlement

As described in Section 6.3 "Future Settlement", we have estimated future total settlement of the existing levees in Reach G to be less than 13 mm (½ inch). This is the result of ongoing consolidation settlement of the foundation clay layers as a result of the 1963, 1983 and 1995 levee construction and fill placement activities.

We have also estimated additional settlement at MSW1W, MSW2E, MSW3E and MSW4E due to the localized fill as high as 1.3 meters (4.3 feet) to be placed for maintenance ramps, as shown in the following table.

Wall	Station	Cross Section Used in Analysis	Max. New Fill Height (m)	Estimated Time Rate of Settlement (mm)				Ultimate Settlement from New Fill (mm)	90% Consolidation (mm)
				1 month	3 months	6 months	1 year		
MSW1W	W40+500	#10	1.3	35	53	67	76	83	75
MSW2E	E21+525	#11	1.3	25	36	46	53	57	52
MSW3E	E26+034 to E26+859	#25	1.2	65	66	66	66	81	73
MSW4E	E27+295 to E27+650	#26	0.8	27	41	54	64	71	64

As shown above, it is estimated that 83 mm (3¼ inches) of settlement will occur at MSW1W, and 57 mm (2¼ inches) of settlement will occur at MSW2E.

9.3.8 Leveling Pad

In order to provide a uniform bearing pad and sliding resistance (along the soil block-foundation interface), we recommend soil be subexcavated to a depth of 0.3 meter (1 foot) below the bottom of the leveling pad and the subexcavation be backfilled with the same backfill material listed in Section 9.3.2 "Materials" ($\phi \geq 34°$) and used within the reinforced-soil block. The subexcavation area should extend laterally at least 0.3 meter (1 foot) on both sides of the leveling pad.

9.3.9 Earthquake Considerations

Each MSE wall structure consists of horizontal wire mesh/strips placed between granular layers. In cross section, it is a rectangle filled with granular soil placed within and next to the clay levee. These materials will perform differently during an earthquake. We

believe that some cracking will occur at or adjacent to the interface between the MSE sand and the lean clay. Consequently, the District should be prepared to repair these cracks after major earthquakes.

9.4 CORROSION INVESTIGATION

An assessment of the potential for corrosion of various buried foundations and structures was performed by Villalobos & Associates. The results of their investigation are presented in Appendix F.

In summary, the field resistivity tests and laboratory testing indicate the following:

- The tested water soluble chlorides ranged from 13 to 15 ppm. These levels are considered to be below the threshold of corrosion
- The tested water soluble sulfates ranged from 139 to 658 ppm. These levels are considered to be negligibly corrosive to moderately corrosive.
- The pH of the tested boring samples ranged from 7.1 to 7.4. These levels are considered to be in the neutral range.
- The levels of bicarbonate ions present in the samples ranged from 281 to 342 ppm. High concentrations of bicarbonates tend to decrease soil resistivities. Bicarbonates are not aggressive to buried steel or concrete, but the lower resistivity environments can promote corrosion activity.

Based on the soil corrosivity data and review of the project requirements, corrosion monitoring and control are recommended to minimize corrosion of metallic and reinforced structures. These recommendations are in accordance with Caltrans Bridge Design Specifications, Section 8.22. The quality of materials and workmanship should be inspected during construction by a corrosion engineer.

Prestressed Concrete Piles

Prestressed concrete piles should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 51 mm (2 inches) should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

Buried Metallic Structures

Based on the low soil resistivity it is recommended that buried metallic piping systems, such as steel and ductile iron, be coated with a bonded dielectric or mortar coating. All non-welded joints should be bonded to provide electrical continuity. Cathodic protection should be installed to provide corrosion protection. Buried metallic piping should be

electrically isolated from other structures, or where a change in piping or coating material occurs.

Buried Reinforced Concrete Structures

Buried reinforced concrete structures should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 76 mm (3 inches) should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

MSE Retaining Walls

For permanent structures, the reinforcement elements shall be designed to have a minimum design life of 75 years. For the metallic reinforcement, galvanization loss will be:

- 15 μm/year/side for the first 2 years
- 4 μm/year/side for subsequent years

The loss of bare steel after the galvanized zinc is depleted will be 12 μm/year/side. These estimates are from the AASHTO Standard Specifications for Highway Bridges, Section 5.8.6.1. It is assumed that the soil backfill used is nonaggressive (i.e. Class 1 Backfill), meeting the following criteria:

- pH of 5 to 10
- Resistivity of not less than 3,000 ohm-cm
- Chlorides not greater than 100 ppm
- Sulfates not greater than 200 ppm.

The above mentioned thicknesses are not valid if the following conditions are present:

- The MSE wall is exposed to marine or other chloride rich environments
- The MSE wall will be exposed to stray currents such as from nearby underground power lines or adjacent railways

The backfill material is aggressive

Section 10

10.1 BACKGROUND

As discussed previously in Section 1.1 "Project Description", CH2MHILL has selected a tieback wall at the following locations, listed in the table below.

Location	Wall Height		Approximate Maintenance Road Elevation (meter)
	Meter	feet	
Montague Expressway West Levee	4.0	13.1	3.1 to 3.3
Highway 101 East Levee	3.5	11.5	5.8 to 6.3
Highway 101 West Levee	3.5	11.5	7.3 to 8.9
Airport Parkway West Levee	3.5	11.5	8.1 to 9.0

At each of these four locations, the tieback wall will be placed in close proximity to an existing pile supported bridge abutment. The purpose of these walls is to allow excavation of the toe of the slope and placement of the maintenance road between the bridge abutment and closest pier (bent). We understand that each wall will have multiple levels of tiebacks.

10.2 SUBSURFACE CONDITIONS

10.2.1 Montague Expressway West Levee

The subsurface conditions at this tieback wall were investigated with Borings WB-07 (at top of West Levee) and CB-07 (in channel).

At Boring WB-07, gravel and sand fill was encountered to a depth of about 0.9 m (3 feet); deeper lean clay fill extended to a depth of about 5.0 m (16.5 feet). The lean clay fill is very stiff to hard except for a soft pocket at a depth of about 4.3 m (14 feet). Underlying the fill are generally layers of medium silt and lean clay to the terminal depth. However, interbeds of medium dense silty sand and poorly graded sand were encountered at depths of 8.2 m (27 feet) and 10.7 m (35 feet), respectively. Based on the cross section drawings provided by CH2MHILL, the wall and tiebacks will be positioned in the lean clay fill and underlying native silt and lean clay.

At Boring CB-07, the ground surface corresponds approximately to the bottom of the proposed wall. Underlying fill deposits consisting of stiff clayey gravel and lean clay fill to a depth of 1.1 m (3.5 feet) are layers of native lean clay and sandy lean clay to a depth of 10.1 m (33 feet). Consistency of the native clay layers is very stiff to a depth of about 1.8 m (6 feet) and decreases to medium to stiff with increasing depth. A very dense silty sand layer underlies the sandy lean clay. By comparison, this sand layer is deeper than the deep sand layer encountered in Boring WB-07.

10.2.2 *Section Not Used*

10.2.3 Highway 101 East Levee

The subsurface conditions at this tieback wall were investigated with Borings EB-25 (closest), EB-26 (about 80 m from bridge) and CB-09 (in channel). Although Borings EB-25 and EB-26 were located at the top of the east embankment, Boring EB-25 is closest to the wall and considered to be the most representative of the tieback and wall subsurface conditions. We have also reviewed Log of Test Borings (LOTB) provided by Caltrans for the Guadalupe River (Widen) project for Highway 101; a copy of the LOTB, along with the General Plan for the widening project, are attached in Appendix G as reference.

Based on Borings EB-25 and EB-26, the embankment fill consists of mostly medium to stiff lean clay with gravel to a depth of about 3.4 m (11 feet) below ground surface (bgs). Underlying the fill are generally layers of stiff to very stiff lean clay, but occasionally medium, to the terminal boring depth of 13.6 m (44½ feet). Interbeds of medium dense to dense silty sand occurred in Boring EB-25 at depths of 8.8 m (29 feet), 11.0 m (36 feet) and 13.3 m (43.5 feet). Based on the cross section drawing provided by CH2MHILL, the wall and tiebacks will be positioned in the lower portion of medium lean clay fill and native medium to stiff lean clay.

At Boring CB-09, the ground surface corresponds approximately to the bottom of the proposed wall. At this location, medium dense clayey gravel and stiff sandy lean clay fill were encountered to a depth of about 1.8 m (6 feet) bgs; it contained pieces of concrete. Underlying the fill are alternating layers of native stiff lean clay, medium dense silty sand, soft silty clay, loose to medium dense clayey sand, medium dense gravel, medium lean clay and stiff silt to the terminal boring depth.

Groundwater was encountered in Boring CB-09 at a depth of 0.3 m (1 foot).

10.2.4 Highway 101 West Levee

The subsurface conditions at this tieback wall were investigated with Borings WB-23 (about 50 m from the bridge), WB-29 (about 30 m from bridge) and CB-09 (in channel). We have also reviewed Log of Test Borings (LOTB) provided by Caltrans for the Guadalupe River (Widen) project for Highway 101 (Bridge No. 37-0037, EA 4874G1); a copy of the LOTB, along with the General Plan for the widening project, are attached in Appendix G as reference.

Based on Borings WB-23 and WB-29, the embankment fill consists of mostly medium to stiff lean clay with gravel to a depth of about 2 m (6½ feet) below ground surface (bgs). Underlying the fill are generally layers of stiff to very stiff lean clay to approximately 4.6 m (15 feet). Below the stiff lean clays are medium lean clay to sandy lean clay with interbedded silty sand and clayey sand lenses extending to the terminal boring depth of 13.6 m (44½ feet). Interbeds of silty sand are loose to dense and occurred in Boring WB-29 at a depth of 6.6 m (21½ feet) and in Boring WB-23 at a depth of 9.1 m (30 feet). Based on the cross section drawing provided by CH2MHILL, the wall and tiebacks will be positioned in the lower portion of the stiff native lean clay or the dense clayey sand.

At Boring CB-09, the ground surface corresponds approximately to the bottom of the proposed wall. At this location, medium dense clayey gravel and stiff sandy lean clay fill were encountered to a depth of about 1.8 m (6 feet) bgs; it contained pieces of concrete. Underlying the fill are alternating layers of native stiff lean clay, medium dense silty sand, soft silty clay, loose to medium dense clayey sand, medium dense gravel, medium lean clay and stiff silt to the terminal boring depth.

Groundwater was encountered in Boring CB-09 at a depth of 0.3 m (1 foot).

10.2.5 Airport Parkway West Levee

The subsurface conditions at this tieback wall were investigated with Borings WB-25 (at top of west levee) and CB-11 (in channel).

At Boring WB-25, stiff lean clay with gravel fill extended to a depth of about 2.9 m (9.5 feet) bgs. Beginning at this depth and extending to a depth of 5.1 m (17 feet) bgs, is a layer of native stiff fat clay. This fat clay has a liquid limit of 80 and a plasticity index of 45. Underlying the fat clay are layers of generally stiff sandy lean clay and lean clay to the terminal boring depth. However, interbeds of medium dense well graded sand and silty sand were encountered between depths of 8.8 m (29 feet) and 11.0 m (36 feet) bgs. Based on the cross section drawing provided by CH2MHILL, the wall and tiebacks will be positioned mostly in the native fat clay and underlying sandy lean clay.

At Boring CB-11, the ground surface corresponds approximately to the bottom of the proposed wall. Underlying fill deposits consisting of loose clayey sand and loose to dense clayey gravel to a depth of 3.5 m (11.5 feet) are layers of native medium dense silty sand and poorly graded sand to a depth of 9.1 m (30 feet). By comparison the well graded sand and silty sand was encountered in Boring WB-25 at a similar depth, but it was relatively thin (2.1 m versus 5.8 m thickness). Below a depth of 9.1 m, a layer of medium lean clay was encountered to the terminal boring depth.

Groundwater was encountered in Boring CB-11 at a depth of 1.5 m (5 feet).

10.3 DESIGN PROPERTIES

The geotechnical design properties for the various soil layers at each tieback wall location are summarized below.

10.3.1 Montague Expressway West Levee

Elevation (meters)	Material*	γ_t (kN/m³)	C' (kPa)	ϕ' (degrees)	C (kPa)
8.5 to 7.6	Clayey Sand Fill	21.2	0	34	--
7.6 to 3.6	Lean Clay Fill	20.4	0	33	192
3.6 to 2.6	Lean Clay Fill	18.1	22	24	41
2.6 to -2.2	Lean Clay	19.6	7	30	36
-2.2 to -3.2	Silty Sand	21.2	0	34	--
-3.2 to -5.1	Lean Clay	18.9	2	30	48

*Boring WB-07

10.3.2 *Section Not Used*

10.3.3 Highway 101 East Levee

Elevation (meters)	Material*	γ_t (kN/m³)	C' (kPa)	ϕ' (degrees)	C (kPa)
12 to 8.6	Lean Clay Fill	20.4	0	33	48
8.6 to 6.4	Lean Clay	18.9	2	30	72
6.4 to 4.5	Lean Clay	20.4	7	30	41
4.5 to 1.8	Silty Sand	21.2	0	35	--
1.8 to -1.1	Lean Clay	21.2	7	30	24

*Boring EB-25 and Caltrans Boring O1-2 (Route 101 at Guadalupe River, January 2002; see Appendix G)

10.3.4 Highway 101 West Levee

Elevation (meters)	Material*	γ_t (kN/m³)	C' (kPa)	ϕ' (degrees)	C (kPa)
11.9 to 9.9	Lean Clay Fill	20.4	0	33	115
9.9 to 7	Lean Clay	19.6	2	30	86
7 to 5.3	Lean Clay/Clayey Sand	19.6	2	30	29
5.3 to 4.3	Silty Sand	19.6	0	30	--
4.3 to -1.2	Lean Clay	21.2	7	30	41

*Boring WB-29

10.3.5 Airport Parkway West Levee

Elevation (meters)	Material*	γ_t (kN/m³)	C' (kPa)	ϕ' (degrees)	C (kPa)
14.3 to 11.2	Lean Clay Fill	18.1	22	24	57
11.2 to 9.1	Fat Clay	18.1	5	20	48
9.1 to 5.5	Lean Clay	20.4	2	30	48
5.5 to 3.3	Silty Sand	21.2	0	36	--
3.3 to 1.3	Lean Clay	18.9	2	30	38

*Boring WB-25

10.4 TIEBACK ANCHOR FEASIBILITY

The feasibility of installing permanent ground anchors for tieback wall structures was evaluated for the five proposed bridge locations (Montague Expressway West and East, Highway 101 West and East, and Airport Parkway West levees) using Federal Highway Administration (FHWA, 1984) design standards and Caltrans standards (1990). We understand the United States Army Corps of Engineers (Corps) has no published design

standards for tieback walls. The FHWA recommends bond lengths of tieback wall anchors not be located in organic soils or in soils with a liquid limit (LL) greater than 50 or a liquidity index (LI) greater than 0.2.

At four locations (Highway 101 East and West, Montague Expressway East and West) the soils behind and below the proposed walls are mostly native lean clays with occasional gravel and clayey sands. At Montague Expressway East the tiebacks might extend into a saturated silty sand layer. At Highway 101 West the tiebacks will likely extend into a clayey sand layer; however, this clayey sand exhibits shear strength characteristics closely resembling the lean clay deposits because of the high percentage of clay content. Therefore, at these locations, it is our opinion tiebacks can be installed within the native lean clay materials, consistent with FHWA standards. Nevertheless, the Contractor should be made aware of brick fragments in fill encountered in Boring WB-25 at a depth of about 1.8 to 2.7 meters (typical) (6 to 9 feet); if other larger debris is encountered at this location at the Airport Parkway West Levee, it might cause installation difficulties. However, this appears to be a localized condition, since it was encountered in only one boring.

At the Airport Parkway West Levee, the native fat clay soils between depths of approximately 2.9 meters and 5.1 meters (9.5 and 17 feet) have liquid limit greater than 50 (actual is 80). Therefore, we recommend the grouted anchor portion of the tieback avoid these upper fat clay soils. Based on the subsurface conditions encountered, we recommend that tieback anchors be inclined at a maximum inclination of 15 degrees. At the Airport Parkway West Levee, this inclination will minimize the potential for penetration of the native fat clay soils underlying the fill approach embankment. A schematic section of this recommendation is shown in Figure 102.

As shown in Figure 103 for the tieback wall at East Levee at Highway 101, we anticipate a potentially liquefiable silty sand layer between approximately Elevation 4.5 to 1.8 meters. To minimize the effect of post earthquake liquefaction settlement, we recommend that tiebacks be terminated in the clay layer overlying the silty sand deposit. Nevertheless, as the result of a strong earthquake, some permanent movement of the tiebacks should be anticipated requiring repairs to the tieback and tieback wall connection.

At the west abutment of Highway 101, we believe the clayey sand layer between approximately Elevation 7.3 and 3 meters is potentially liquefiable. The likely consequence of liquefaction in this area is post-liquefaction settlement less than 75 mm (3 inches). A more detailed discussion of liquefaction potential affecting the performance of the levee is presented in Section 11. At this wall location, we recommend the tiebacks be designed with the guidelines presented on Figure 104. These guidelines assume the wall is designed for static and seismic loading conditions without liquefaction, and that the District would make necessary repair if damage occurs due to liquefaction. The following reasons help formulate our recommendations:

1) Based on a discussion with Mr. Claudio Avila, Caltrans Geotechnical Engineer for the design of the northbound stem bridge for the Highway 101 widening project, we understand that Caltrans do not expect lateral spreading or adverse effect on the performance of the abutment due to liquefaction of the embankment foundation soils,

if any, and therefore has elected not to improve the clayey sand layer to reduce the potential of liquefaction.

2) The probability of a strong earthquake coinciding with a 100-year flood event is low.

Alternatively, the potentially liquefiable sand layer could be improved with techniques such as compaction grouting. This probably can be achieved with vertical grout holes spaced at 2 to 3 m (6 to 9 feet) and at a grid pattern extending laterally beyond the bond zone as shown in Figure 104. This approach will require traffic control, including reroutes and most probably night work since Highway 101 is an important transportation route for the South Bay region.

10.5 ANCHOR LENGTHS

The length of the tieback wall anchors can be estimated based on earth pressures, spacing of the tiebacks, diameter of anchor hole and soil grout adhesion.

As shown on Figure 105, tieback wall anchors in cohesive soils should be designed such that the bond zone is located behind the minimum free length plane. A minimum of 4.6 m (15 feet) of free length (measured from the back of the wall) is recommended for all anchors. Additionally, the bond zone of the anchors should be located behind the potential failure surface. The potential failure surface should be assumed to be a line parallel to the Rankine active plane which extends from the base of the wall and is offset away from the wall by a distance of 1.5 meters (5 feet) or 1/5 the height of the wall, whichever is greater. The Rankine active plane should be assumed to be located at an angle of 45 degrees from the plane of the wall for cohesive soils encountered at this site. A minimum cover of 4.6 meters (15 feet) is also recommended for the bond zone of the anchor. Also, in order to minimize any effect on the existing bridge abutment piles, it is recommended that tieback anchor zones be located a minimum of 1.5 meters (5 feet) behind the pile locations as determined from Caltrans As-built Plans. The composite of these criteria were then used to determine an appropriate location for the anchor-soil bond zone for the West Levee at Airport Parkway as shown on Figure 102.

In general, the recommended tieback wall anchor bond lengths in cohesive soils range from a minimum of 4.6 meters (15 feet) to a maximum of 12.2 meters (40 feet). Tiebacks are generally inclined at 15 to 45 degrees from the horizontal. For this project, an inclination of 15 degrees maximum from the horizontal is recommended to provide anchor bond zones. It has been our experience an anchor diameter of 152 to 305 mm (6 to 12 inches) is typically used in the Santa Clara Valley area.

10.6 LATERAL EARTH PRESSURES

10.6.1 Static Loading Conditions

Lateral Earth Pressures

Lateral earth pressures of the embankment are anticipated to be high due to the relative stiffness of the tieback system, the cohesive nature of the native materials, and the

surcharge resulting from the slope of the retained embankment. We have estimated the design earth pressure using the following equation:

$$p = 0.8\ K_a\ \gamma'\ H'\ (\text{psf})$$

where:
- p = design earth pressure, psf
- K_a = coefficient of active earth pressure based on the Coulomb theory
- γ' = effective unit weight of retained soil, pcf
- H' = refer to Figures 106 and 107 for definition, feet

Schematics of the pressure distribution are presented in Figures 106 and 107. These pressure distribution are based on Caltrans Memo to Designer 5-12, "Earth Retaining Structures Using Tiebacks" dated August 1990, and a project communication with Mr. Emil Vergara, Caltrans, in 1997.

The above design earth pressure equation is based on Caltrans Memo to Designers 5-12. We understand that Caltrans assumes the long term cohesion approaches zero, and ϕ' (angle of internal friction) approaches a finite value between 26 and 34 degrees for clay soils. Based on the plasticity index values and unit weights for the clay soil encountered, we have estimated the soil properties, design earth pressure and H′ value for each location. Those values are summarized in the following table.

Location	γ (pcf)	ϕ' (degrees)	K_a	H (feet)	H (m)	H′ (feet)	H′ (m)	P (psf)
Montague Expressway West Levee	125	29	0.32	13.1	4.0	14.0	4.3	30 H′
Highway 101 East Levee	125	29	0.39	11.5	3.5	19.5	5.9	40 H′
Highway 101 West Levee	125	29	0.39	11.5	3.5	19.5	5.9	40 H′
Airport Parkway West Levee	115	26	0.39	11.5	3.5	21.5	6.6	40 H′

We have assumed no traffic surcharge for the above recommended values.

The wall pressures recommended above are based on the assumption that no differential hydrostatic pressures will develop behind the walls. The buildup of hydrostatic pressure behind the wall should be prevented by the inclusion of a positive drainage system. A manufactured prefabricated drainage membrane, such as Miradrain 6000, could be used behind the wall. In the event that hydrostatic pressure builds up, the submerged unit weight (γ') should be used. Then, the pressure distribution as shown on Figure 107 should apply.

Positive drainage for water collected in the backfill should be provided by means of perforated drain pipe leading to a free drainage outlet or by weep holes extending through the walls.

Bridge Abutment Loading

At each bridge where tieback walls are planned, the adjacent existing abutment is supported on piles. Consistent with Caltrans practice, we recommend that each tieback wall be designed to resist the lateral load capacity of each existing pile. The magnitude of the design lateral pile load will depend on the allowable lateral deflection, to be determined by the Structural Engineer. No pile size (diameter) was shown on the As-built drawings for the Highway 101 Bridge. Assuming that all existing piles are driven concrete and greater than 305 mm (12 inches) in width or diameter, we recommend that the lateral load from each pile be 58kN (13 kips, Caltrans Bridge Design Specifications, January 1993) for a horizontal deflection of 6 mm (1/4 inch); this lateral load should be positioned at a depth of 1.2 meters (4 feet, Caltrans private correspondence, 1997) below bottom of bridge abutment pile cap. If a design horizontal deflection of 25 mm (1 inch) is selected, we understand Caltrans design criteria requires 178 kN (40 kips) per pile if the tieback walls are located within a 45-degree angle from the top of the piles. These pile loads should be added to the static loads discussed in Section 10.6.1 "Static Loading Conditions."

Surcharge Loading

Another lateral load imposed on each tieback wall is induced by vehicular surcharge assumed to be equal to 0.6 meter (2 feet) of soil behind the wall. The uniform lateral pressure, q, on the full wall height is as follows:

$$q = 2K_a\gamma \text{ (psf)}$$

$$q = (2)\ (125)\ K_a = 250\ K_a \text{ (psf)} = 12.0\ K_a \text{ (kPa)}$$

Since K_a varies, we recommend the surcharge pressure in the following table.

Location	K_a	q (kPa)	q (psf)
Monterey Expressway West Levee	0.32	3.83	80
Montague Expressway East Levee	0.32	3.83	80
Highway 101 West Levee	0.39	4.67	98
Highway 101 East Levee	0.39	4.67	98
Airport Parkway West Levee	0.39	4.67	98

10.6.2 Dynamic Loading Conditions

We understand that Caltrans customarily does not require that retaining walls be designed for increased lateral earth pressures resulting from earthquake ground motion. This design approach considers that the static earth pressures are conservatively selected and should provide sufficient margin of safety against increased lateral pressure due to earthquake shaking. For a seat type abutment supported on piles, Caltrans Memo to Designer also requires the lateral load exerted on the piles be transferred to the tieback walls (see above section on "Bridge Abutment Loading"). According to CH2MHILL, this addition of the lateral pile loads becomes the controlling case for the seat type

abutment tieback walls. Based on this practice, Caltrans has reported that retaining walls generally have performed well after earthquakes (i.e., no apparent distress). For example, retaining walls on Caltrans highways received a good structural performance report following the January 17, 1994 Northridge earthquake.

10.7 SOIL-GROUT BOND

Assuming the tiebacks are grouted within the existing medium to very stiff cohesive materials with pockets of soft clay, ultimate soil/grout adhesion values between 23.9 and 47.9 kPa (500 and 1,000 psf) are typical. Recommended design soil grout adhesion values is presented in the table below for each of the four wall locations. Since the adhesion value and the strength of the soil/grout bond are influenced by the construction method, anchor capacities should be verified by proof testing each anchor following installation. We recommend a safety factor of 2.5, consistent with FHWA criteria.

Location	Design Soil Grout Adhesion (psf)
Montague Expressway West Levee	750
Highway 101 East Levee	1,000
Highway 101 West Levee	1,000
Airport Parkway West Levee	1,000

10.8 BEARING CAPACITY

We understand the base of the tieback walls will be subject to a downward (compressive) load component from the inclined tiebacks. Consequently, a continuous strip footing is planned. Spread footings, when supported on new or existing engineered fill material consisting of stiff to very stiff clay soils, should be designed such that soil bearing pressures do not exceed the values summarized in the table below.

LOCATION	Design Bearing Pressures					
	Dead Load Only		Dead Load plus Live Load		Dead plus Live plus Earthquake/Wind Loads	
	psf	kPa	psf	KPa	psf	kPa
Montague Expressway West Levee	1,000	47.88	1,500	71.82	2,000	95.76
Highway 101 West Levee	1,500	71.82	2,000	95.76	2,500	119.70
Highway 101 East Levee	1,500	71.82	2,000	95.76	2,500	119.70
Airport Parkway West Levee	1,500	71.82	2,000	95.76	2,500	119.70

10.9 FRICTION BETWEEN BACK OF WALL AND SOIL

We understand a portion of the downward (compression) load from the inclined tiebacks will be resisted by friction between the concrete (back of wall) and soil. For clayey backfill, this frictional resistance is estimated in terms of a uniform adhesion value. The following table summarizes the recommended ultimate values (safety factor of 1.0) of the clay to concrete adhesion.

Location	Clay to Concrete Adhesion	
	psf	kPa
Montague Expressway West Levee	750	35.91
Highway 101 West Levee	800	38.30
Highway 101 East Levee	800	38.30
Airport Parkway West Levee	850	40.70

10.10 SETTLEMENT OF SOLDIER PILES

Although structural details are not currently available, we assume vertical soldier piles will be used. As discussed in Section 9.5 "Anchor Lengths", the tiebacks will be inclined at an angle of approximately 15 degrees from the horizontal. Since the tiebacks will be permanently tensioned, this will result in a horizontal and vertical (downward) load component transferred into the individual soldier piles. The horizontal component will be resisted by the soldier pile and wall. The downward force at the bottom of the soldier piles should not exceed the recommended soil pressures. If desired, we would be pleased to estimate settlement of the soldier piles and develop recommended soil pressures when structural loads and soldier pile layout are available.

10.11 TYPES OF SOLDIER PILES

Soldier piles might consist of either reinforced concrete or steel. They are either installed with pile driving equipment or are set in pre-excavated holes and then concreted in place. If steel piles are used, double channels are recommended to eliminate eccentric loading (i.e., torque) on the soldier piles.

10.12 MINIMUM SPACING OF TIEBACKS

The recommend minimum tieback spacing within the grouted (anchor bond) zone, centerline to centerline, is 3 hole diameters or 1.2 meters (4 feet), whichever is less.

10.13 CORROSION INVESTIGATION

An assessment of the potential for corrosion of various buried foundations and structures was performed by Villalobos & Associates. A summary of the investigation is presented in Section 9.4. The detailed results of Villalobos & Associates investigation are presented in Appendix F.

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to the cyclic loading associated with earthquake ground shaking. Based on the latest ground motions anticipated for this site (Section 2, "Site Geology and Seismicity"), we have updated the evaluation of liquefaction potential performed previously for our April 2000 report. The current evaluation was based on semi-empirical methods considering the following three fault systems: 1) a moment magnitude 8.0 earthquake on the San Andreas fault, 2) a moment magnitude 7.1 on the Hayward fault and 3) a moment magnitude 6.5 on the Hayward Southeast Extension fault. The basis of these earthquake magnitudes and site accelerations is described in the Section 2, "Site Geology and Seismicity."

The clay layers at the site are not susceptible to liquefaction. Therefore, the potential for liquefaction of various sand and gravel layers at the site was evaluated. On the basis of our analysis, it appears that there is some potential for liquefaction to occur at the locations shown on Figure 2, Location of Potentially Liquefiable Layers. The potentially liquefiable layers appear to be concentrated in the areas near the San Jose International Airport, upstream of Highway 101, although a few scattered potentially liquefiable layers exist along the remainder of these reaches of the levee alignment. These layers range in thickness from 0.9 to 6.1 meters (3 to 20 feet). Should liquefaction occur, settlement could occur at these locations. Settlement of the ground surface is estimated to be on the order of 12 to 102 mm (½ to 4 inches) at these locations. Estimates of these potential settlements were made using the procedures outlined by Tokimatsu and Seed, (1984).

Based on explorations conducted, most layers of potentially liquefiable soil were encountered below depths of at least 3 meters (10 feet) below ground surface at landside toe. These deposits are overlain by layers of non-liquefiable cohesive soils. Empirical research by Ishihara (1985) indicates that for level ground, liquefaction is not likely to result in visible damage at the ground surface as long as the cover of the liquefiable soil is equal to at least 3 meters (10 feet). Based on Ishihara's research and our local experience, we expect ground surface rupture is unlikely.

We performed post-liquefaction slope stability analysis at two of the more critical cross-sections (Cross-Sections 23 and 24) using residual strengths estimated for the potentially liquefiable sand layers. The factors of safety against undrained loading simulating conditions immediately after liquefaction are both above 1.3. Therefore, we conclude that lateral spreading is unlikely at the liquefaction prone locations in these reaches. The results of the above analysis are presented below.

Cross Section			Factor of Safety End of Construction			Residual Undrained Shear Strength
Number	Station	Levee	Minimum Required	Riverside	Landside	Psf
23	11+250	East	1.3	1.5	1.9	60
24	11+790	West	1.3	1.5	2.2	60

12.1 GENERAL

All site preparation and earthwork should be done under the observation of a representative of the Geotechnical Engineer and in accordance with the recommendations presented herein. In general, grading/earthwork will include raising crown and side slopes of existing levees, constructing a new levee, excavating and backfilling for new floodwalls, excavating for new tieback walls, constructing depressed maintenance roads and ramps and removal of sediment in the channel.

12.2 CLEARING AND DEMOLITION

All concrete flatwork, existing pavements, aggregate base, armoring aggregate, loose stone, drainage lines, underground utilities, trees, brush, vegetation, fencing and abandoned structures designated for removal should be demolished and removed from the site. Abandoned utilities encountered during grading should be removed in their entirety. Existing Geotextile Soil Reinforcing (GSR) walls are located on the crown of the following levees: the East Levee north of Route 237, on the West Levee between Gold Street and Route 237, on the East Levee north and south of Tasman Drive and on the West Levee north of Tasman Drive. According to the 1995 As-built drawings, semi-impervious or structural backfill ($PI \leq 25 \pm 5$) was placed inside the outermost 1.5 to 2.7 meters (5 to 9 feet) landside portion of the GSR. If left in place, there is the potential for horizontal seepage, particularly through the semi-impervious material. Therefore, these GSR walls should be removed in their entirety.

12.3 GRUBBING

Grubbing consists of the removal, within the levee foundations, floodwalls, maintenance road and ramp areas, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectional matter. Roots or other intrusions over 38 mm (1½ inches) in diameter within the levee foundations, floodwalls and maintenance road areas should be removed to a depth of 0.91 m (3 feet) below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation areas. Materials resulting from grubbing operations should be removed from the site. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill consisting of material similar to adjoining soils, should be placed in layers up to the final foundation grade.

12.4 STRIPPING

After foundation clearing and grubbing operations are complete, stripping should be commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil, including but not limited to the side slopes and crown of existing levees, the landside and waterside toe of existing levees, and the footprint of new levees and maintenance roads and ramps. The depth of stripping should be determined by local conditions and is expected normally to vary from 152 to 305 mm (6 to 12 inches). The final depth of stripping should be determined by a representative of the Geotechnical

Engineer during site preparation. Materials resulting from stripping operations should be removed from the site. Organic topsoil that does not contain debris may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. The stripped materials should not be re-used as engineered fill or blended with other materials.

12.5 EXPLORATION TRENCH

An exploration trench (often termed "inspection trench") should be excavated under all new levees, including the new Highway 101 Setback Levee (West Levee) planned just north of Highway 101. The purpose of this trench is to expose or intercept any undesirable underground features such as old drain titles, water or sewer lines, abandoned wells, animal burrows, buried logs, pockets of unsuitable material, or other debris. The trench should be located at or near the centerline of hauled fill levees. Dimensions of the trench will vary will soil conditions and embankment configurations. Backfill should be placed only after a careful review of the excavation trench by a representative of the Geotechnical Engineer to confirm that seepage channels or undesirable materials are not present; if they are, they should be excavated with a base of sufficient width to allow backfill compaction with conventional compaction equipment. To backfill narrower trenches properly, special compaction procedures and/or equipment would be required. Trenches should have a minimum depth of 1.83 meters (6 feet) except for embankment heights less than 1.84 meters (6 feet), in which case the minimum depth should equal the embankment height.

12.6 REMOVAL OF SEDIMENT IN CHANNEL

The channel sediment areas should be excavated as shown on the drawings to bring those areas to their finish channel elevations. All sediment materials should be removed from the site and should not be re-used for construction of levees, maintenance roads, ramps and backfill for floodwalls. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect.

12.7 EXCAVATION

After the areas to receive fill have been properly cleared, grubbed and stripped, it is recommended that existing loose fills or soft deposits be removed to the underlying native soils and replaced with engineered fill. Floodwall and tieback wall areas should be excavated as required to bring those areas to their finish subgrade elevations. All loose soil should be removed from the exposed floodwalls and tieback walls subgrade prior to wall construction.

12.8 SUBGRAGE PREPARATION

After demolition, stripping and subgrade excavations are complete, the exposed surface to be filled should further be prepared by scarifying the top 152 mm (6 inches) of the surface, moisture conditioning the material to near the optimum content, and

recompacting to a minimum relative compaction of 90 percent of its maximum dry density in accordance with ASTM Test Designation D 1557. This procedure is intended to provide a tight bond between the foundation and fill, and to eliminate a plane of weakness at the interface. The foundation surface should be kept drained and not scarified until just prior to fill placement in order to minimize the risk of saturation from rainfall. The base of the exploration trench (just north of Highway 101) should also be scarified, moisture conditioned and compacted prior to the placement of new fill materials. The fill soils exposed in footing excavations for floodwalls and tieback walls are not expected to require scarification. However, the final need for scarification should be determined at the time of excavation.

At some locations, such as at toe of existing levees or depressed maintenance roads, the subgrade surfaces will be positioned near or below the free groundwater level or in areas which have been subjected to standing water. The exposed materials are expected to consist of low plasticity clays, silts and fine sands.

Control of the water in most shallow excavations in these materials probably could be accomplished by sloping them to drain and pumping from sumps within the excavated areas. After dewatering, some of the subgrade surfaces might be stable enough to support construction equipment; whereas, other areas might develop pumping or rutting. Please refer to Section 12.11 "Construction Dewatering", for further discussion regarding this subject.

In order to minimize the potential for pumping subgrade conditions, it is recommended that the first lift of the fill be "track-walked" in place to a thickness of about 0.3 meter (1 foot) using a bulldozer or other track-mounted equipment. The initial lift should be dry of the laboratory optimum and compacted to a minimum relative compaction of 90 percent. Rubber tired equipment, such as scrapers or bottom dump trucks, should not be permitted to traverse the area. Pumping or rutting will be further exacerbated by repeated passes of rubber tired equipment. This could occur wherever a silt or clay subgrade is encountered in the existing levee fill or native soil.

In the event that deep seated pumping develops, it is recommended that the unstable subgrade areas be "bridged" using a combination of stabilization fabric (Mirafi 600X or equivalent) covered by a layer of granular, bridging material. The bridging material should consist of "bank run" or "crusher run" material conforming to the gradation discussed in Section 12.9.3.2 "Bridging Material." Since bridging material is highly permeable, seepage should be expected within this layer. Therefore, bridging material should only be used in areas where seepage is acceptable.

After the reinforcing fabric has been placed on the exposed subgrade surface, the bridging material should be track-walked into place over the fabric. It is estimated that a 0.46 to 0.61 meter (18 to 24 inch) thick layer of bridging material probably will be needed. Rubber tired equipment should not be permitted to traverse pumping areas until placement of the bridging material and stabilization fabric have been completed. Then additional fill can be placed and compacted to provide desired finished grades.

In areas where the silt or clay subgrade pumps or ruts (in levee fill or native soil), lime treatment is an alternative treatment to achieve stabilization. Since lime stabilization results in a relatively impervious soil, it can be used anywhere in the levee cross sections.

Where subgrade areas are silty sand or clayey sand, they may require a combination of quicklime and flyash stabilization. The Contractor should retain his own testing laboratory to perform tests to determine the appropriate lime and/or flyash content; this should include unconfined compressive strength tests. It has been our experience on other local projects that 4 to 5 percent quicklime by weight will generally achieve the desired stabilization results. All lime/flyash stabilized subgrade should be prepared in accordance with Section 24 "Lime Stabilization" of the July 1992 Caltrans Standard Specifications. The Contractor should retain a Specialty Subcontractor experienced with lime stabilization. Furthermore, the Contractor should include this work in his bid price.

12.9 FILL MATERIALS

12.9.1 Approval

Soil materials, whether from sources on or off site, should be approved by a representative of the Geotechnical Engineer for the intended use and specifically for a required location or purpose. It is expected that most materials will be imported.

12.9.2 Onsite Materials

Channel Sediment

Where it is planned to remove portions of the channel (sediment), these materials should not be re-used as engineered fill. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. Otherwise, these materials should be removed from the site.

Excavation for Floodwalls, MSE Walls, Wingwalls, and Tieback Walls

The existing levee is engineered fill composed primarily of stiff to hard compacted lean clay with occasional fat clay. Where is it is planned to remove portions of the existing levee for new floodwall foundations and new tieback walls, the materials could be reused as new levee fill providing they are properly processed to achieve a moisture content near the optimum prior to placement. The drying of materials with excessive moisture content should be in accordance with the requirements in Section 9.9 "Fill Placement and Compaction". However, excavation for the tieback walls, will require some selectivity to be exercised if these materials are to be considered for re-use as possible new levee fill. Where excavations for tieback walls are made into existing clay fill, these materials may be re-used; whereas if these excavations extend deeper into organic soils or other objectionable material, these materials should not be re-used as engineered fill.

Geotextile Soil Reinforcing Walls

Where Geotextile Soil Reinforcing (GSR) walls are encountered, these walls should be removed in their entirety, to eliminate the potential for horizontal seepage. These materials should be hauled offsite.

Existing Armoring Aggregate

The existing surficial layer of armoring aggregate (on the crown, i.e. crest), should be removed. If left-in-place, this horizontal layer of aggregate material could allow seepage through and onto the landside slope. This material could be re-used for surfacing new maintenance roads, provided it is not contaminated or mixed with fine-grained clay or silt soils.

12.9.3 Import Materials

General Levee Fill

It is recommended import materials used for levee construction consist of a soil or soil-rock mixture free from organic matter or other deleterious substances. The material should contain at least 50 percent by weight of clayey soils (passing the U.S. Standard No. 200 sieve). The clayey soils should be of low to medium plasticity as classified by the Unified Soil Classification System having a plasticity index within the range of 10 to 20 and a liquid limit less than 40. The material should not contain rocks or lumps over 152 mm (6 inches) in greatest dimension and should not contain more than 15 percent greater than 64 mm (2½ inches). Sand and gravel size particles should comprise no more than 50 percent of the soil-rock mixture. It is anticipated that materials meeting these general requirements are available from local quarries or borrow pits located offsite.

Select Levee Fill

Select levee fill for the outer shell of levee with slope inclination 1.5 to 1 (horizontal to vertical) or steeper than 2 to 1 should consist of material free from organic material and substantially free of shale or other soft, poor durability particles. It should not contain slag aggregate or recycled materials, such as glass, shredded tires, portland cement concrete rubble, asphaltic concrete rubble, or other unsuitable material as determined by the Engineer. The material should contain no more than 30 percent by weight of clayey soils (passing the U.S. Standard No. 200 sieve). The clayey soils should be of low to medium plasticity as classified by the Unified Soil Classification System having a plasticity index within the range of 10 to 20 and a liquid limit less than 40. The material should not contain rocks or lumps over 152 mm (6 inches) in greatest dimension and should not contain more than 15 percent greater than 64 mm (2½ inches). Sand and gravel size particles should comprise a minimum of 70 percent of the soil-rock mixture. It is anticipated that materials meeting these general requirements are available from local quarries or borrow pits located offsite. In addition, the laboratory compacted samples of the fill material should have a minimum effective internal friction angle of 36 degrees

based on consolidated undrained triaxial tests with pore pressure measurements or drained triaxial tests.

Bridging Material

Bridging material, for bridging soft subgrades for maintenance road areas, should consist of a reasonably well graded mixture of coarse angular gravel and cobbles. The following gradation limits are recommended as a general design guideline.

Sieve Size		Percentage Passing
(mm)	(inches)	
152	6	100
51	2	0 – 50
19	¾	0 – 10
Durability Index: 25 minimum		

Armoring Aggregate

Armoring aggregate for placement on the crown of the levees, maintenance roads and ramps, should consist of clean, hard and durable gravel or crushed rock, conforming to the following gradation.

Sieve Size		Percentage Passing
(mm)	(inches)	
76	3	100
38	1½	75-55
19	¾	60-40
13	½	45-30
10	3/8	35-20
6	¼	25-15
No. 200	No. 200	20-0

The rock for armoring aggregate should have a minimum specific gravity of 2.40 in accordance with California Test Method No. 206 and a minimum durability index of 35 in accordance with California Test Method No. 229.

12.10 FILL PLACEMENT AND COMPACTION

All levee fills should be spread in uniform horizontal lifts not exceeding 203 mm (8 inches) in uncompacted thickness. Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either aerating the material if it is too wet, or spraying the material with water if it is too dry. The lean clay or occasional fat clay should be compacted in the range from optimum to 3 percent wet of

laboratory optimum. The native granular materials should be compacted near the laboratory optimum. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of water content. To prevent drying of the subgrade soils, placement of fill should start immediately after surface preparation and should proceed in a continuous operation until the site is brought to grade. No fill should be placed during rain or when saturation will hinder proper compaction. The finished grade surface of the compacted fill under floodwall and tieback wall footings should be kept in a moist condition prior to the placement of concrete. Jetting or flooding of the fill should not be permitted.

Relative compaction is the ratio of the in place dry density of the constructed fill to a maximum dry density determined by ASTM Test Designation D 1557. All levee fills should be compacted to a minimum relative compaction of 90 percent, as determined by ASTM Test Designation D 1557. All pavement aggregate base course, subbase and subgrade materials, and any fill material which is within the upper 600 mm (2 feet) below finished pavement surface should be compacted to a minimum relative compaction of 95 percent. All fill material below the bottom of footings and all other structural elements should be compacted to a minimum relative compaction of 95 percent. All other fill materials should be compacted to a minimum relative compaction of 90 percent. A representative of the Geotechnical Engineer should be present to observe all grading operations during both preparation of the site and compaction of engineered fill.

The compaction and moisture content of the subgrade and each layer of levee placed should be tested by a representative of the Geotechnical Engineer in accordance with ASTM D 1556 "Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method" or ASTM D 2922 "Standard Test Method for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)", ASTM D 3017 "Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)" and ASTM D 2216 "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock."

Fills placed on slopes steeper than 6:1 (horizontal to vertical) should be benched into the adjacent soils to provide a bond at the interface. Loose soils on the surface of existing slopes should be excavated as new fill is placed. Benching is expected to be necessary where new fill will be placed on existing levee slopes in order to raise the existing levee. It is recommended that horizontal benches be constructed such that the maximum height of the vertical section of the bench does not exceed 305 mm (1 foot).

Where new fill is placed on the face of an existing levee, the fill should extend laterally a minimum of 0.61 meter (2 feet) beyond the finished lines during construction. After compaction, the excess fill should be removed to the final grade and lines.

12.11 CONSTRUCTION DEWATERING

Because the groundwater level is below the bottom of the anticipated depth of excavations for floodwalls, MSE walls, wingwalls, tieback walls, and the "inspection" or "exploration" trench for levee just north of Highway 101, the excavation bottoms are expected to be relatively dry during the excavation phase. In the event that water

becomes ponded during the rainy season, the ponded water could be removed by sump pit/pump procedures.

Depending on the season of the year, the water level could vary considerably in the Guadalupe River channel. It is conceivable that some riverside maintenance road subgrade areas may be soft and wet. As discussed previously, those areas should be over-excavated and bridging material placed to a maximum depth of 0.61 meter (2 feet); alternatively, lime stabilization treatment could be used. The water encountered could be removed by sump pit/pump procedures.

Section 13

13.1 GENERAL

Based on our site reconnaissance, as well as our subsurface investigation and geotechnical analyses, we believe the geotechnical condition of the existing East and West Levees between Route 237 and I-880 as described herein is generally very good. These opinions and our recommendations are discussed below in more detail in this section.

13.2 RESULTS OF LEVEE SLOPE STABILITY EVALUATION

In Section 4 " Levee Slope Stability Evaluation", we discussed (1) the cross sections selected for analysis, (2) method of analysis and (3) stability results. We identified a total of seventeen (17) levee sections as candidates for analysis with side slope of 2H:1V. Based on our slope stability analyses of these 17 sections, we conclude that all of these sections had factors of safety that met or exceeded the minimum factor of safety required for four (4) Corps' design conditions. For the "pseudo-static" case, Cross Sections 19, 20, 23 and 24 have marginal factors of safety, i.e. less than 1.0. However, based on earthquake loading conditions, we anticipate permanent vertical deformation of the levees to be in the range of 30 to 150 mm (1.2 to 6 inches) and about 320 mm (12.5 inches) at Cross Section 23.

In addition, we evaluated the slope stability of fourteen (14) cross sections with a 1.5H:1V landside slope. Results of our analysis indicate that the existing levee fill strength is insufficient to provide a minimum factor of safety required by the Corp for the steady seepage load case. Based on our parametric evaluation of the material strength, we conclude that portions of the levee will have to be remove and reconstructed with a select quality material with a minimum internal friction angle (ϕ') of 36 degrees in order to meet the Corp's design criteria. We recommend that the outer shell of the slope with a minimum width of 2.4 m, extending down to the toe of the slope, should be replaced with select material.

As previously stated, it is unlikely that the design earthquake event will coincide with a period of high water. However, all levees should be inspected following a major seismic event since deformations due to earthquake shaking are anticipated. Any levee repairs and/or upgrades should be consistent with the District's levee safety program.

13.3 RESULTS OF SETTLEMENT ANALYSES

In Section 6 "Settlement Evaluation", we discussed (1) land subsidence, (2) inferred levee settlement, (3) predicted settlement and (4) future levee settlement. The latter is based on consolidation of the foundation clay soil due to previous filling in 1963, 1983 and 1995, as well as filling due to proposed levee raise. South of Station 14+700 settlement at Cross Sections 29 and 30 is nil since these areas were not filled and will not be filled. The future settlement north of Station 14+700 is estimated to be on the order of 6 to 46 mm (¼ to 1¾ inches) for the levees except Cross Section 24 where future settlement is estimated to be on the order of 62 mm (2½ inches). Levees should be overbuilt to accommodate this future settlement.

In Section 9.1.3 "Wall Settlement," the estimated time to achieve 90% consolidation for floodwalls is about 9 to 24 months. If it is desired to construct the floodwalls before the 9 month period, then a surcharge fill with a minimum height of 1.5 meters (5 feet) is recommended. The surcharge settlement periods are listed in detail in Section 9.1.3.

We estimate maximum differential settlement of the levee and floodwall (placed on spread footings) to be about 12 mm (½ inch) in a distance of 30 meters (100 feet). If this estimated settlement is acceptable to the Structural Engineer and the District, then spread footing foundations are recommended. The floodwall should be overbuilt to accommodate this future settlement.

If these settlements are not acceptable to the Structural Engineer and the District, then drilled piers are recommended for the support of the floodwalls. Drilled piers should extend below the compressible foundation clay layers, however. In addition, a concrete cutoff wall should extend a minimum of 1 meter (3.3 feet) below lowest adjacent finished grade. Since the levee will settle relative to the floodwall, periodic grouting may be required to fill any voids between the wall foundation and levee fill soil.

13.4 RESULTS OF SEEPAGE ANALYSES

In Section 7 "Seepage Evaluation" we discussed (1) method of analysis and (2) cross sections and results of analysis. In general, the levees are constructed of well compacted lean clays with some localized fat clays. Furthermore, most of the native soils underlying the levee fills are cohesive. Since the cohesive soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be low. To confirm these opinions, we performed a seepage analysis on seventeen cross sections.

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee at 17 cross sections is expected to be less than 1/10 gallon per minute (gpm). In our opinion these seepage quantities are very small.

The above discussion is for a steady state condition such that seepage through the levee achieves equilibrium during the 100-year flood elevation. To simulate the change in river level during the flood event, we also performed a transient analysis based on the 100-year flood hydrograph. Based on this analysis, the highest phreatic surface is at 48 hours; this surface is lower than the steady state phreatic surface. By comparison, we conclude that the estimated seepage quantities probably will be less than the estimated values at the 16 cross sections.

As discussed in Section 1.1 "Project Description", Geotextile Soil Reinforcing (GSR) walls were constructed in 1995 north and south of Tasman Drive on the East Levee and north of Tasman Drive on the West levee. This resulted in raising the levees by a height of 0.76 to 1.77 meters (2.5 to 3.83 feet) for a distance of about 910.7 meters (2,987 feet). According to the 1995 As-built drawings, semi-impervious or structural backfill (PI≤25±5) was placed inside the outermost 1.5 to 2.7 meters (5 to 9 feet) landside portion of the GSR. We believe it would be prudent to remove these GSR walls in their entirety,

to eliminate the potential for horizontal seepage, particularly through the semi-impervious material.

We also recommend that prior to raising the crest of any existing levee, the surficial layer of armour aggregate be removed. If left-in-place, this horizontal layer of aggregate material could allow seepage through and onto the landside slope.

13.5 SITE PREPARATION AND GRADING

In Section 12 "Site Preparation and Earthwork" we discussed in detail the following earthwork aspects: (1) clearing and demolition, (2) grubbing, (3) stripping, (4) exploration trench, (5) removal of sediment in channel, (6) excavation, (7) subgrade preparation, (8) fill materials, including onsite materials and import materials, (9) fill placement and compaction and (10) construction dewatering. Please refer to that section for details.

13.6 LIQUEFACTION POTENTIAL

In Section 11 "Liquefaction Potential," we presented our updated evaluation of liquefaction potential of the site and concluded that there are locations that have a relatively high potential of liquefaction during major earthquake shaking. These locations are presented graphically in Figure 2. However, it is our opinion, based on residual strength post-liquefaction stability analysis, that lateral spreading due to liquefaction is unlikely. The likely consequence that may impact the levees will be liquefaction induced settlement; we estimate such settlement to be on the order of 12 to 102 mm (½ to 4 inches). Therefore, we recommend that the District inspect the levee after major earthquake events to assess the need for raising the settled crest of the levee and/or grouting underneath floodwalls if gaps result from the levee settlement. In any event, the inspection and mitigation should be consistent with the District's Levee Safety Technical Guidance Manual.

Section 14

The conclusions and recommendations contained in this report are based on exploratory borings drilled for this study, as well as our review of available subsurface information, our previous site reconnaissance, and our local experience and engineering judgment. Furthermore, the slope stability analysis has been limited to seventeen (17) critical cross sections and fourteen (14) 1.5H:1V landside slope cross sections.

Surveyed ground elevations of borings were not available. Elevations were estimated from nearby cross section geometry provided by the District and from spot elevations provided by CH2MHILL.

An investigation for subsurface environmental contamination was beyond the scope of our services.

The recommendations presented in this report were developed with the standard of care commonly used in this profession. No other warranties are included, either express or implied, as to the professional advice presented in this report.

Section 15

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Table 1. Bridge Locations and Analysis Comments
Lower Guadalupe River Flood Control Project

Bridge	Levee	Approximate Station	Assumptions/Comments
UPRR	East	7+008	No depressed maintenance road – no analysis necessary
UPRR	West	7+008	No depressed maintenance road – no analysis necessary
Gold Street	East	7+120	No depressed maintenance road – no analysis necessary
Gold Street	West	7+120	No depressed maintenance road – no analysis necessary
Route 237	East	8+020	Addressed in Task 2.4.1
Route 237	West	8+056	Addressed in Task 2.4.2
Tasman Drive	East	9+260	2:1 cut slope based on 10/30 workshop drawings, concrete paving on slope
Tasman Drive	West	9+260	2:1 cut slope based on 10/30 workshop drawings, concrete paving on slope
Montague Expressway	East	11+843	On hold, alternatives being evaluated by District – No analysis at this time. Please Advise
Montague Expressway	West	11+843	On hold, alternatives being evaluated by District – No analysis at this time. Please Advise
Trimble Road	East	13+700	Completed by the City of San Jose – no analysis necessary
Trimble Road	West	13+700	Completed by the City of San Jose – no analysis necessary
Highway 101	East	14+525	Tieback wall based on 10/30 workshop drawings
Highway 101	West	14+525	No changes - no analysis necessary.
Airport Island	East	14+700	Stepped gabions
Airport Island	West	14+700	No changes – no analysis necessary
Airport Parkway	East	15+700	
Airport Parkway	West	15+700	No changes – no analysis necessary
Skyport Drive	East	16+300	Completed by the City of San Jose – no analysis necessary
Skyport Drive	West	16+300	Completed by the City of San Jose – no analysis necessary
I-880	East	17+600	Completed by Corps of Engineers – no analysis
I-880	West	17+600	Completed by Corps of Engineers – no analysis

TABLE 2
Lower Guadalupe River Flood Control Project
EXISTING BORING LOG INDEX LIST

Boring Index	Consultants	Consultants' Project Number	Project Date	Project Name
AA-n	Woodward-Clyde	973061NA	Sept 1997	Preliminary Investigation, Fuel Storage and Dispensing Facility for American Airlines, San Jose
BSP-n	Woodward-Clyde	4327	Dec 1960	Soil Investigation for the Proposed Bay Shell Plant Storage Tanks, San Jose
GJJ-n	Gribaldo,Jacobs, Jones & Associates	E584-M1	Jan 1963	Soil Investigation for Guadalupe River Project, Reach No. 1, Preliminary Study of the West Bank of Alviso Slough and Proposed SPRR Retaining Wall, Santa Clara County
GRI-n	John V. Lowney & Associates	109-11 PA 9669	Jan 1981	Geotechnical Investigation, Guadalupe River Improvement, San Jose
GRIM-n	Woodward-Clyde	15482V	Oct 1982	Geotechnical Investigation, Guadalupe River Flood Control Improvements, Southern Pacific Railroad to the County Marina in Alviso
GRIV-n	Woodward-Clyde	4246	c. 1960	Soil Investigation for the Proposed Guadalupe River Project, Reaches 1 and 2, Santa Clara County
GROCS-n	John V. Lowney & Associates	109-12 PA 11516	June 1983	Geotechnical Investigation, Guadalupe River Outfall Control Structure, San Jose
GSB-n	Woodward-Clyde	16338V	Oct 1984	Geotechnical Investigation, Raising of Gold Street Bridge over the Guadalupe River in Alviso, San Jose
GSBOLD-n	Woodward-Clyde	4246	c. 1960	Gold Street Bridge Investigation as part of the 1960 Guadalupe River Project
GTPP-n	Parikh Consultants, Inc.	953009NE	c. 1996	Golden Triangle Pipeline Project, South Bay Water Recycling Program
HHP-n	Woodward-Clyde	91C0639R	Jan 1992	"Hetch-Hetchy Pipelines," Relocation of San Francisco Water Department, Bay Division Pipeline Numbers 3 and 4 at Guadalupe River and Coyote Creek, San Jose and Santa Clara
HHPA-n	PSC Associates, Inc.	88122.10	July 1988	Lick Mill Blvd. Extension, Santa Clara
OPS-n	Woodward-Clyde	16784V	c. 1985	Geotechnical Consultation, Oakmead Storm Pump Station Outfall Spillway

TABLE 2
Lower Guadalupe River Flood Control Project
EXISTING BORING LOG INDEX LIST

Boring Index	Consultants	Consultants' Project Number	Project Date	Project Name
PRMAR-n	Soil Mechanics & Foundation Engineers	1140	Feb 1962	Soil Engineering Report for the Proposed Marina, Alviso, Santa Clara County
PYH-n	Cooper & Clark	33A	Jan 1960	Preliminary Site Investigation, Proposed Yacht Harbor, Alviso
RBPS-n	Woodward-Clyde	16171X	Feb 1987	Geotechnical Investigation, Retention Basin and Pump Station, San Jose International Airport, San Jose
REAL1-n	Woodward-Clyde	5135	June 1962	Soil Investigation, Guadalupe River Project, Realignment within Reach 1, Santa Clara
RHM-n	Woodward-Clyde	92C0526R	Nov 1992	Soil Investigation, Riparian Habitat Mitigation, Route 87
RREMB-n	John V. Lowney & Associates	109-1 PA 1233	Oct 1969	Soil Mechanics Investigation, SPRR Bridge and Embankment Raising over Guadalupe River, San Jose
SBYC-n	Woodward-Clyde	15482-W-2000	April 1983	Geotechnical Study, South Bay Yacht Club Relocation, Alviso
SDB-n	Woodward-Clyde	963010NA	c. 1997	Geotechnical Investigation, Skyport Drive Bridge, San Jose
SIL-n	Cooper & Clark	170-G	Oct 1982	Soil Investigation, Landslide Areas, Guadalupe River, San Jose
SJIA-n	Woodward-Clyde	16171V	Feb 1984	Geotechnical Investigation, San Jose International Airport, San Jose
SPRR-n	Jo Crosby & Associates	2706	Jan 1976	Soils Investigation for Guadalupe River, SPRR to Trimble Road, Project # 3015
SPRRB-n	Woodward-Clyde	4246	Jan 1961	Soil Investigation for the Proposed Southern Pacific Company Bridge, Alviso (in conjunction with soil investigation of Guadalupe River Project, Reaches 1 and 2)
TDB-n	Woodward-Clyde	16266V	May 1984	Geotechnical Study, Tasman Drive Bridge at Guadalupe River, Santa Clara/San Jose

Note:

1. The value "n" in the boring index column represents the Boring Number used in the corresponding project, such as 1 and B1.

TABLE 3a

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 1

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	**Average**
Monte Vista - Shannon	6.8	R	13.8	15.9	13.8	0.45	0.46	0.48	0.45	**0.46**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	10.6	10.8	10.6	0.46	0.58	0.48	0.56	**0.52**
Calaveras	6.7	SS	13.8	14.0	13.8	0.36	0.42	0.37	0.38	**0.38**
San Andreas - Peninsula Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - 1906 Rupture	8	SS	20	20.1	20	0.38	0.58	0.45	0.58	**0.50**
Sargent	6.8	OR	17.8	19.9	17.8	0.37	0.36	0.40	0.38	**0.38**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 3b

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 2

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	**Average**
Monte Vista - Shannon	6.8	R	14.4	16.4	14.4	0.43	0.45	0.47	0.43	**0.45**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	11.9	12.1	11.9	0.42	0.55	0.45	0.52	**0.48**
Calaveras	6.7	SS	13.1	13.3	13.1	0.38	0.44	0.38	0.39	**0.40**
San Andreas - Peninsula Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - 1906 Rupture	8	SS	20	20.1	20	0.38	0.58	0.45	0.58	**0.50**
Sargent	6.8	OR	17.5	19.6	17.5	0.37	0.37	0.41	0.38	**0.38**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 3c

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 3

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	**Average**
Monte Vista - Shannon	6.8	R	13.8	15.9	13.8	0.45	0.46	0.48	0.45	**0.46**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	13.8	13.9	13.8	0.38	0.50	0.41	0.47	**0.44**
Calaveras	6.7	SS	13.8	13.9	13.8	0.36	0.42	0.37	0.38	**0.38**
San Andreas - Peninsula Segment	7.2	SS	19.5	19.6	19.5	0.31	0.41	0.34	0.39	**0.36**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	19.5	19.6	19.5	0.31	0.41	0.34	0.39	**0.36**
San Andreas - 1906 Rupture	8	SS	19.5	19.6	19.5	0.39	0.59	0.46	0.59	**0.51**
Sargent	6.8	OR	16.9	19.0	16.9	0.38	0.38	0.42	0.39	**0.39**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 3d

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 4

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	**Average**
Monte Vista - Shannon	6.8	R	12.5	14.6	12.5	0.48	0.50	0.52	0.48	**0.49**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	16.3	16.4	16.3	0.34	0.45	0.37	0.42	**0.40**
Calaveras	6.7	SS	13.1	13.3	13.1	0.38	0.44	0.38	0.39	**0.40**
San Andreas - Peninsula Segment	7.2	SS	18.8	18.9	18.8	0.32	0.42	0.35	0.40	**0.37**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	18.8	18.9	18.8	0.31	0.41	0.34	0.39	**0.36**
San Andreas - 1906 Rupture	8	SS	18.8	18.9	18.8	0.31	0.41	0.34	0.39	**0.36**
Sargent	6.8	OR	16.3	18.4	16.3	0.39	0.40	0.43	0.40	**0.40**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 4
Lower Guadalupe River Flood Control Project
SUBSURFACE CHARACTERIZATION
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT

Levee	Levee Section Number	Station Limits		Comments
		From	To	
West	W1	7+000	7+450	
	W2	7+450	8+200	Common with E2
	W3	8+200	8+500	
	W4	8+500	9+230	
	W5	9+230	10+100	
	W6	10+100	11+730	
	W7	11+730	13+000	
	W8	13+000	14+040	
	W9	14+040	14+570	Common with E9
	W10	14+570	15+450	Common with E10
	W11	15+450	15+900	
	W12	15+900	16+770	
	W13	16+770	17+550	
East	E1	6+200	6+640	
	E2	6+640	8+050	Common with W2
	E3	8+050	9+550	
	E4	9+550	10+140	
	E5	10+140	11+100	
	E6	11+100	11+440	
	E7	11+440	12+450	
	E8	12+450	13+650	
	E9	13+650	14+850	Common with W9
	E10	14+850	15+56	Common with W10

Table 5A
Lower Guadalupe River Flood Control Project
PROPOSED 17 CROSS SECTIONS OF LEVEES CONSIDERED TO BE MOST CRITICAL

Cross Section Number	Levee Section Number	Approximate Station	Section Location	Critical Boring(s)	Comments
14	W3	8+280	West Levee	WB-03, GRIV-16	Ditch
15	E3	8+580	East Levee	OPS-3, OPS-4, GRIV-14	Levee raise
16	W4	8+640	West Levee	GRIV-14, WB-04	Previously problematic, ditch on landside, levee raise
17	W4	9+180	West Levee	GRIV-11, WC-03	Levee raise
18	W5	9+600	West Levee	WB-05, GRIV-9	Levee raise, previously problematic, excavation
19	E4	9+780	East Levee	GRIV-8	Levee raise, loose sand
20	E4	10+080	East Levee	HHP-1	Levee raise, excavation
21	W6	10+470	West Levee	WB-06	Levee raise, excavation
22	E5	10+740	East Levee	EB-10	Levee raise, excavation
23	E6	11+250	East Levee	EB-11	Levee raise, loose sand, excavation
24	W7	11+790	West Levee	WB-07	Levee raise, ditch
25	E7	12+120	East Levee	SPRR-1, EC-09	Levee raise, excavation
26	E8	12+810	East Levee	EB-13, EC-10	Levee raise, excavation
27	W8	13+890	West Levee	SJIA-S12, WC-08, WC-07	Levee raise, excavation
28	E9	14+370	East Levee	WB-14, EB-16	Levee raise, excavation
29	E10	15+176	East Levee	SJIA-46, SJIA-45	
30	W13	17+370	West Levee	WB-17	

Table 5B
Lower Guadalupe River Flood Control Project
14 LEVEE CROSS SECTIONS WITH MODIFIED OUTBOARD SLOPE (1.5H:1V)

Cross Section Number	Levee Section Number	Approximate Station	Section Location	Closest Exploration(s)
39	E3	E22+630	East Levee	EB-08, DPS-3, DPS-4
40	E3	E22+830	East Levee	EC-05, GRIV-13
41	E5	E24+240	East Levee	HHP-1, EC-06
42	W6	W43+440	West Levee	GRIV-6, HHPA-3, HHPA-2
43	E5	E25+010	East Levee	EB-30, EB-10
44	W6	W44+290	West Levee	GRIV-4, WC-05
46	E7	E25+670	East Levee	EB-12, EC-07, EC-08
47	W6	W44+860	West Levee	GRIV-2, WB-07
50	E7	E26+190	East Levee	EC-09, SPRR-2
51	W7	W45+350	West Levee	WB-08
52	E7	E26+375	East Levee	EC-09, SPRR-7
53	W7	W45+540	West Levee	WB-08, WB-09
54	E8	E26+940	East Levee	EB-13, EB-14, EC-11, SPRR-4
55	W7	W46+150	West Levee	WB-10, WB-11

TABLE 6
Lower Guadalupe River Flood Control Project
SUMMARY OF EFFECTIVE SHEAR STRENGTH PARAMETERS
BASED ON ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TESTS

SOIL TYPE	DRY DENSITY RANGE (pcf)	ESTIMATED EFFECTIVE STRENGTH PARAMETERS							
		ϕ' (degrees)				c' (psf)			
		Peak	10% ε	20% ε	Design	Peak	10% ε	20% ε	Design
Lean CLAY FILL	Less than 110	23.7	22	16.3	24	456	466	574	450
Lean CLAY FILL	Greater than 110	32.2	33.8	30.1	33	0	0	0	0
Fat CLAY	Less than 80	19.8	12.2	NA	20	113	214	NA	110
Lean CLAY	From 80 to 100	30.4	29	26.7	30	35.1	182.6	186.5	35
Lean CLAY	From 100 to 110	29.4	30	25.3	30	153.4	144.8	424.2	150

Note: Definitions for attached figures are

$q = \frac{1}{2} (\sigma_1' - \sigma_3')$

$p = \frac{1}{2} (\sigma_1' + \sigma_3')$

TABLE 7
Lower Guadalupe River Flood Control Project
SECOND STAGE SHEAR STRENGTH PARAMETERS
3 STAGE RAPID DRAWDOWN SLOPE STABILITY ANALYSIS

Material	Consolidated Undrained Strength Test Results				Second Stage Strength Parameters			
	ϕ_R	c_R	ϕ'	c'	ψ_R	d_R	ψ_S	d_S
	deg.	psf	deg.	psf	deg.	psf	deg.	psf
Fat Clay Fill	12.8	25.5	20	20	15	30	20	20
Lean Clay Fill	13.6	1147	33	60	14	1200	33	60

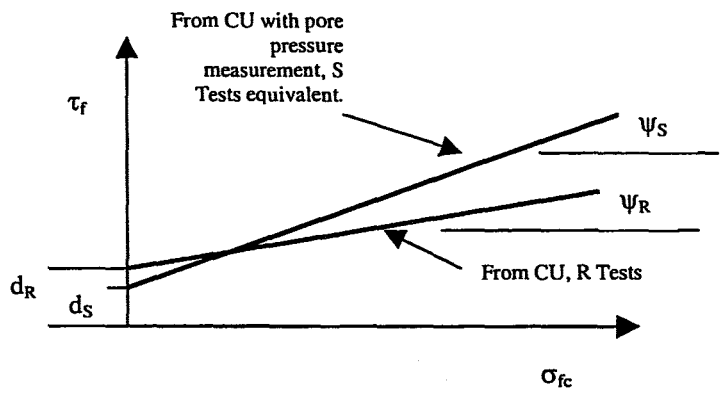


Table 8 Flood Wall Design Parameters

Flood Wall No.	Unit Weight (saturated) kN/m^3	ϕ' degree	c' kPa	Su kPa	Ko (landside)	Ko (riverside)	Kp (horizontal)	Kp (2H:1V slope)	Ultimate Bearing Capacity kPa
FW5E, FW4W	20.7	33	0	67	0.66	0.37	3.2	1.7	287
FW6W	20.3	22	19	67	0.66	0.37	3.2	1.7	287
FW7E, FW8E, FW9W, FW10W	20.3	22	19	67	0.66	0.37	3.2	1.7	287
FW9E, FW10E, FW11W, FW12W	20.6	33	0	67	0.66	0.37	3.2	1.7	287

Table 9 Summary of Planned Inboard Retaining Walls

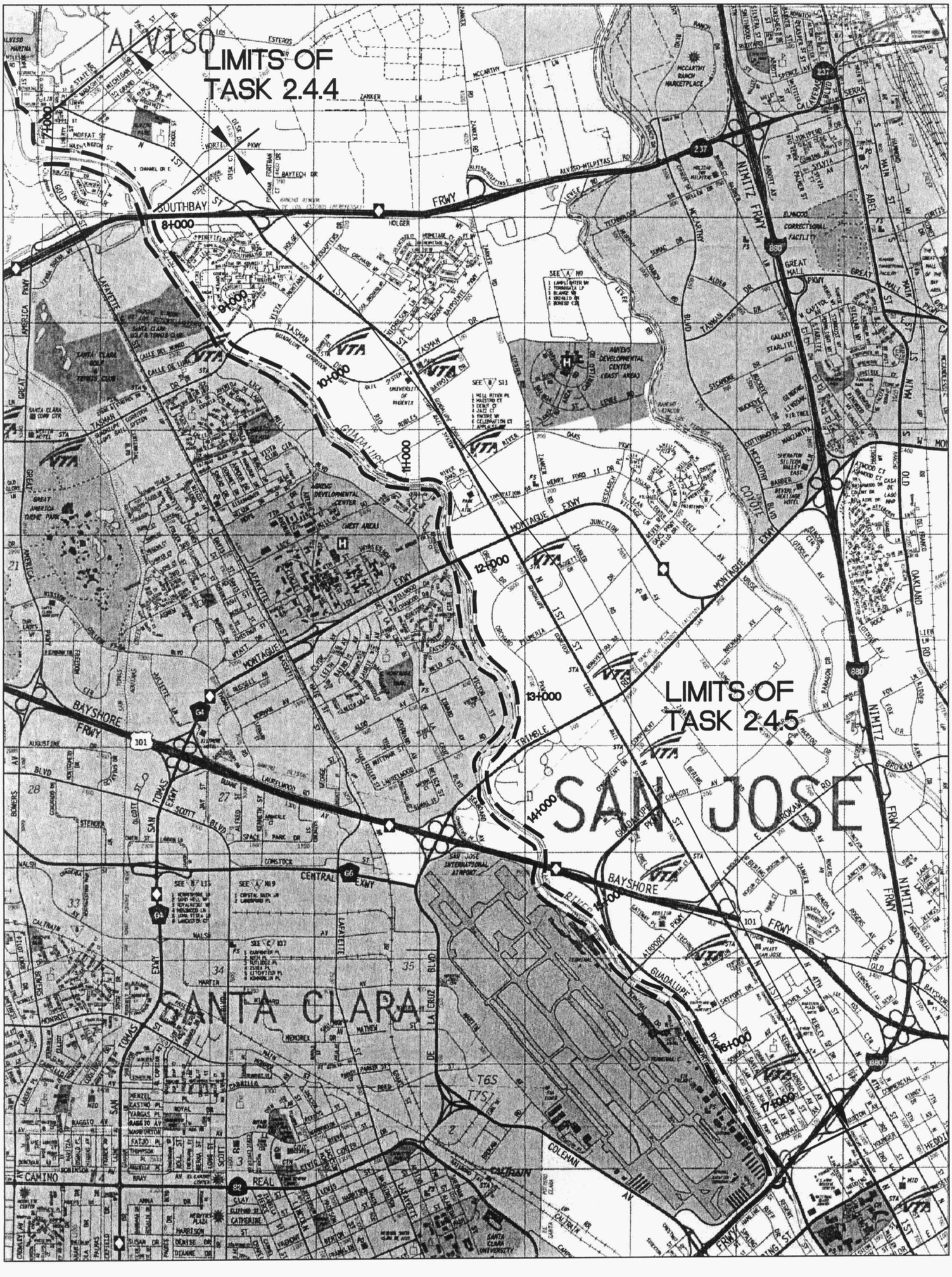
Retaining Wall No.	Location	Type	Outboard/Inboard	Levee	Station From	Station To	Maximum Height (m)
RW3E	Tasman Drive	Type 1	Inboard	East	23+141	23+241	4.3
RW4E	Tasman Drive	Type 1	Inboard	East	23+289	23+392	5
RW6E	Montague Expressway	Tieback & Type 1	Inboard	East	25+733	26+024	4.5
RW3W	Montague Expressway	Tieback & Type 1	Inboard	West	45+057	45+192	3.2
RW5E	Trimble Road	Type 1	Inboard	East	27+810	27+882	
RW7E	Hwy 101	Type 1	Inboard	East	28+579	28+647	3.3
RW4W	Hwy 101	Type 1	Inboard	West	47+684	47+791	4.1

Table 10 Inboard Retaining Wall Design Parameters

(Corp Guideline EM 1110-2-2502 Paragraph 4-4)

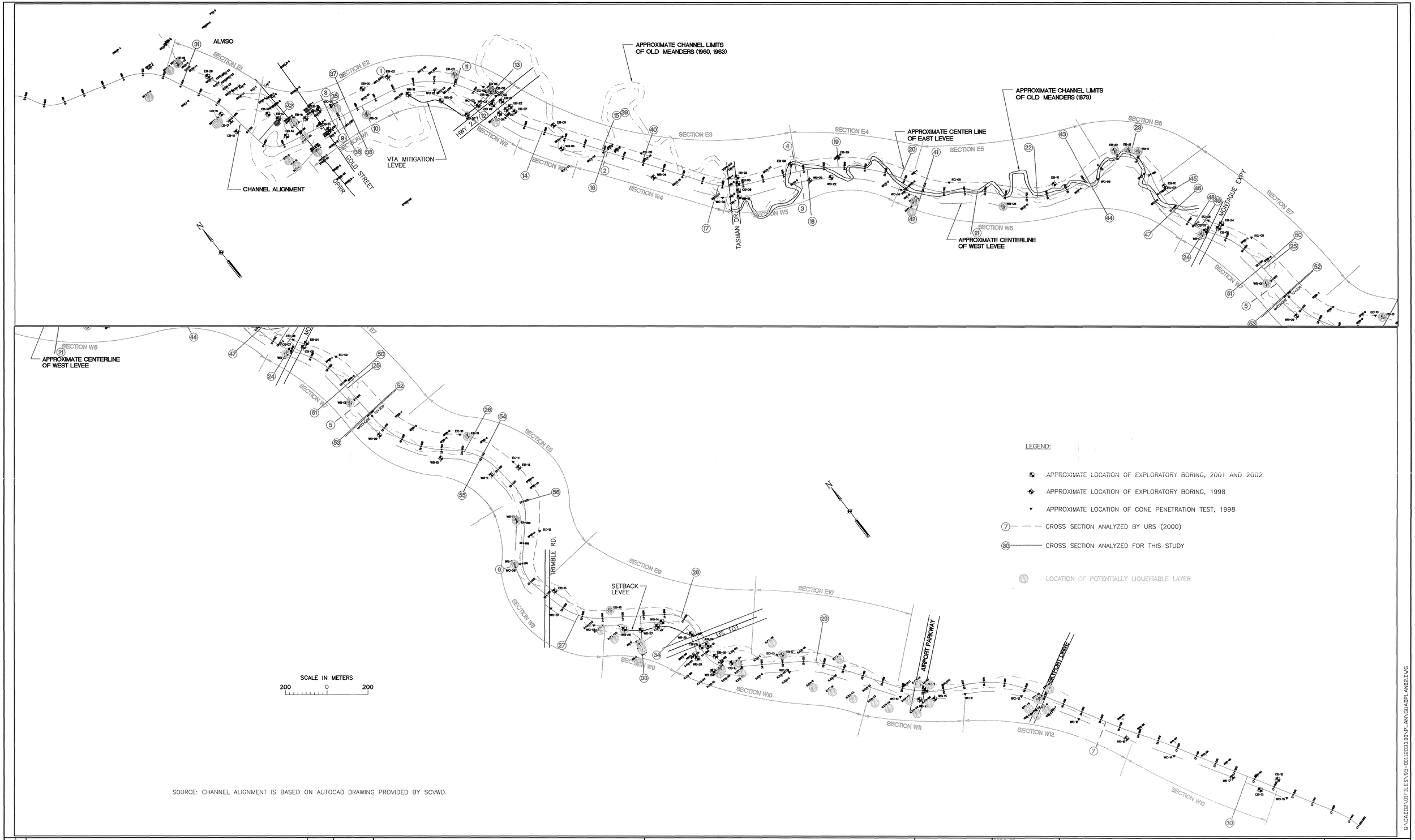
Retaining Wall No.	Location	Unit Weight (saturated)	ϕ'	c'	Su	Ko	Ko	Ko	Kp	Kp	Kp	Ultimate Bearing Capacity	Horizontal Accelera-tion Coefficient k_h
		kN/m^3	degree	kPa	kPa	horizontal backfill	slope backfill at 2H:1V	slope backfill at 1.5H:1V	horizontal	negative slope 2H:1V	Negative slope 1.5H:1V	kPa	
RW3E	Tasman Drive	20.4	26	11	50	0.5	0.6	0.75	3.0	1.1	0.75	287	0.2
RW4E	Tasman Drive	20.4	26	11	50	0.5	0.6	0.75	3.0	1.1	0.75	287	0.2
RW6E	Montague Expressway	18.8	29	12	50	0.52	0.6	0.77	2.85	1.05	0.77	287	0.2
RW3W	Montague Expressway	18.8	29	12	50	0.52	0.6	0.77	2.85	1.05	0.77	287	0.2
RW5E	Trimble Road		33	0	67	0.66	0.37	1.7	0.66	0.37	1.7	287	0.2
RW7E	Hwy 101	18.8	24	14	50	0.5	0.6	0.75	3.0	1.1	0.75	287	0.2
RW4W	Hwy 101	18.8	24	14	50	0.5	0.6	0.75	3.0	1.1	0.75	287	0.2

Figures



NOT TO SCALE

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD PROTECTION PROJECT Santa Clara County, California	SITE MAP	Figure 1
7/18/02	URS		



LEGEND:

- APPROXIMATE LOCATION OF EXPLORATORY BORING, 2001 AND 2002
- APPROXIMATE LOCATION OF EXPLORATORY BORING, 1998
- APPROXIMATE LOCATION OF CONE PENETRATION TEST, 1998
- (7) CROSS SECTION ANALYZED BY URS (2000)
- (30) CROSS SECTION ANALYZED FOR THIS STUDY
- LOCATION OF POTENTIALLY LIQUEFIABLE LAYER

SOURCE: CHANNEL ALIGNMENT IS BASED ON AUTOCAD DRAWING PROVIDED BY SCVWD.

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REV	DESCRIPTION OF REVISION	BY	DATE
5	MOVED LOCATION OF SEVERAL BORINGS AND CPTS AND REVISED NUMBERS.	S.H.	3/19/98
4	MOVED WC-4 AND WB-5 PER CULTURAL RESOURCE RECOMMENDATION.	S.H.	4/9/98
3	ADDED CROSS SECTIONS 8 THROUGH 30 AND LEVEE SECTIONS.	M.L.L.	9/18/01
2	ADDED BORING LOCATIONS (2001 STUDY) AND VTA MITIGATION LEVEE	J.W.	11/05/01
1	ADDED NEW BORING LOCATIONS, SETBACK LEVEE, CROSS SECTIONS 31 THROUGH 56	T.P.	5/29/02
0			

URS
55 South Market Street, Suite 1650
San Jose, Ca. 95113

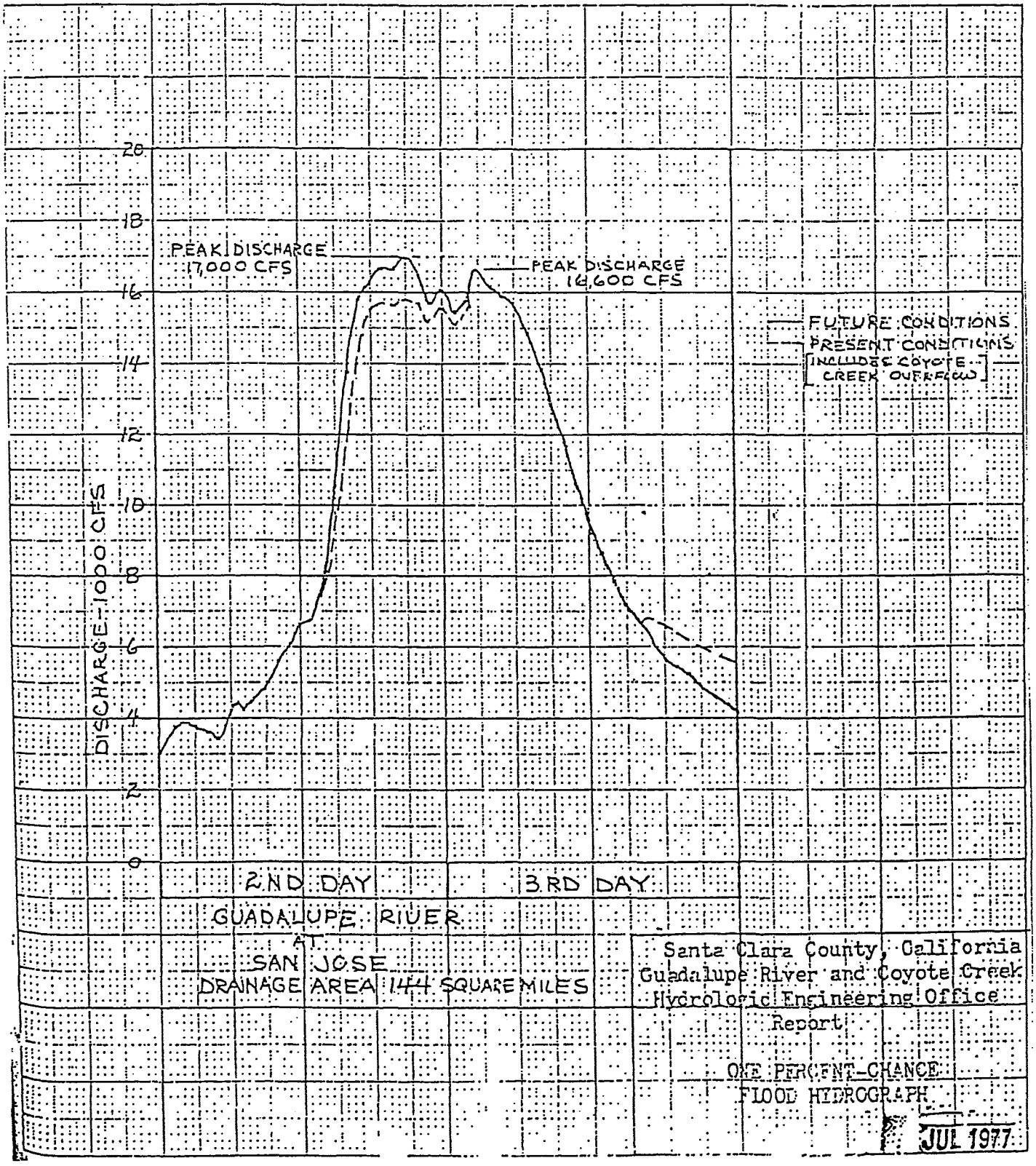
DESIGNED T.P.
DRAWN E.G.
CHECKED S.H.
PEER REVIEWED M.L.
PROJECT MANAGER P.B.
DATE 7/1/02

Lower Guadalupe River Flood Protection Project Santa Clara County, CA.

SITE AND FIELD EXPLORATION LOCATION PLAN

REVISION 5
PROJECT 95-00112030.00
DRAWING Fig. 2
SHEET 1 OF 1

TEMP013



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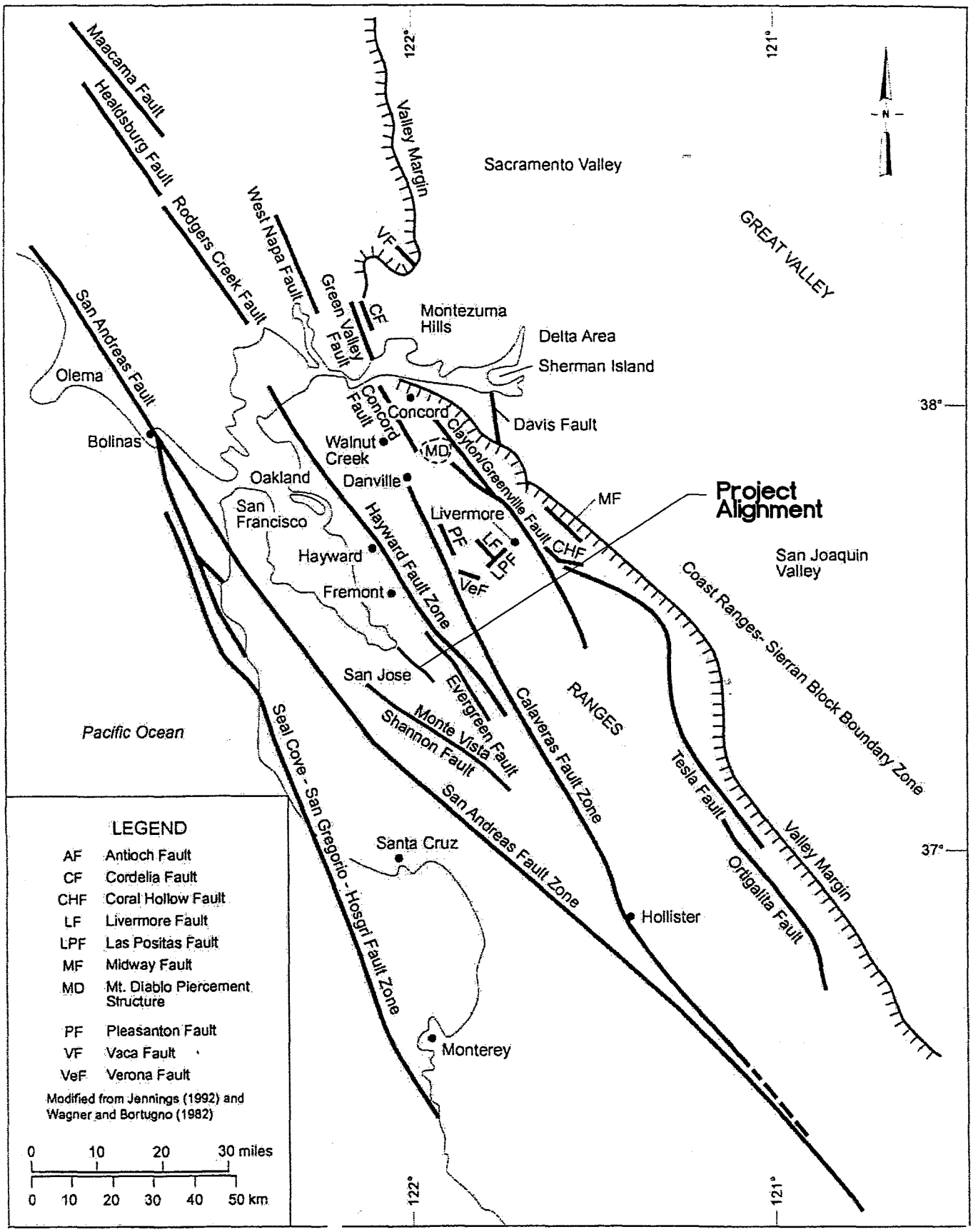
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	ONE PERCENT CHANCE FLOOD HYDROGRAPH	Figure 3
5/31/02			
URS			



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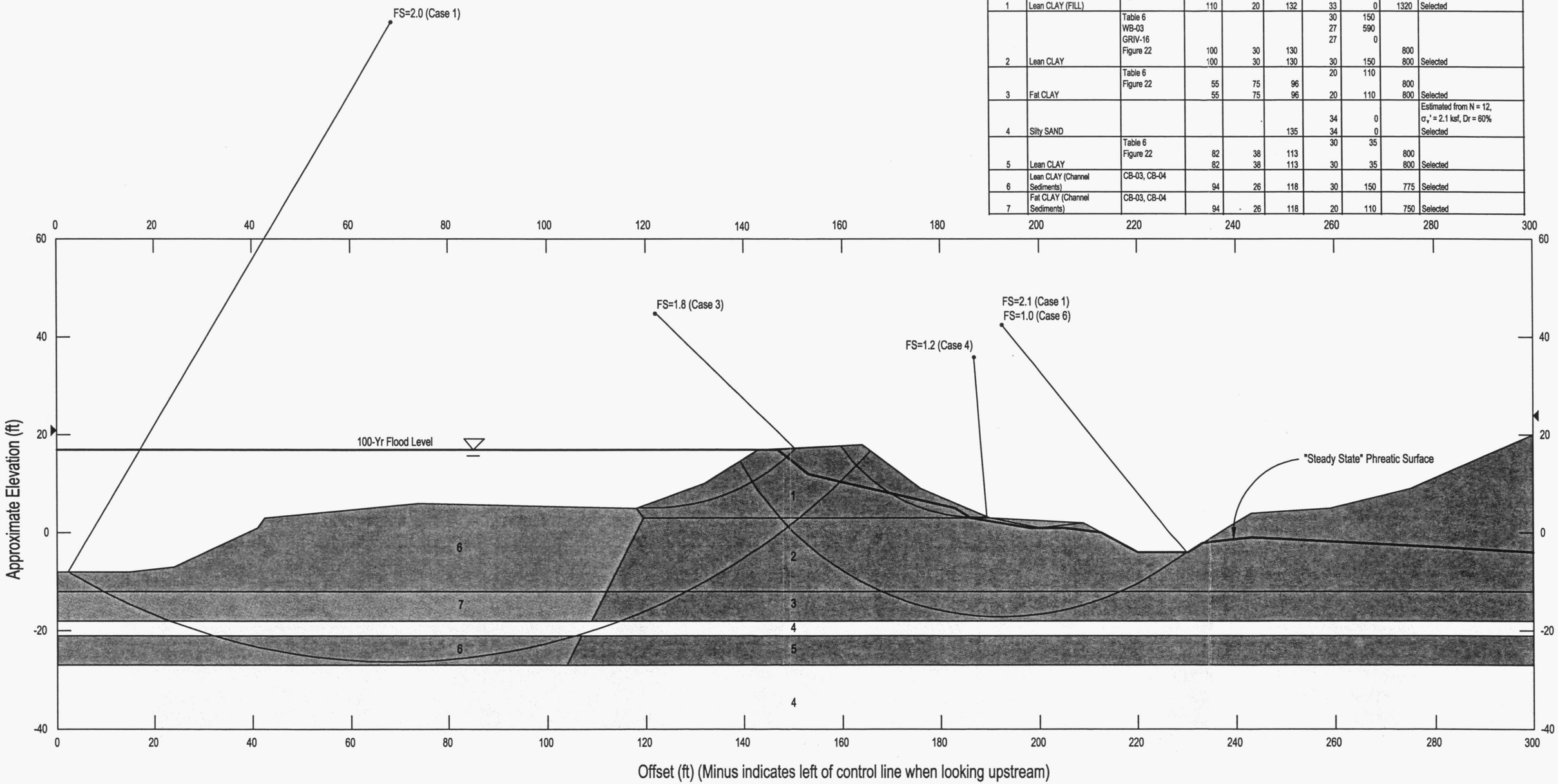
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	REGIONAL ACTIVE FAULTS	Figure 4
7/15/02			
URS			

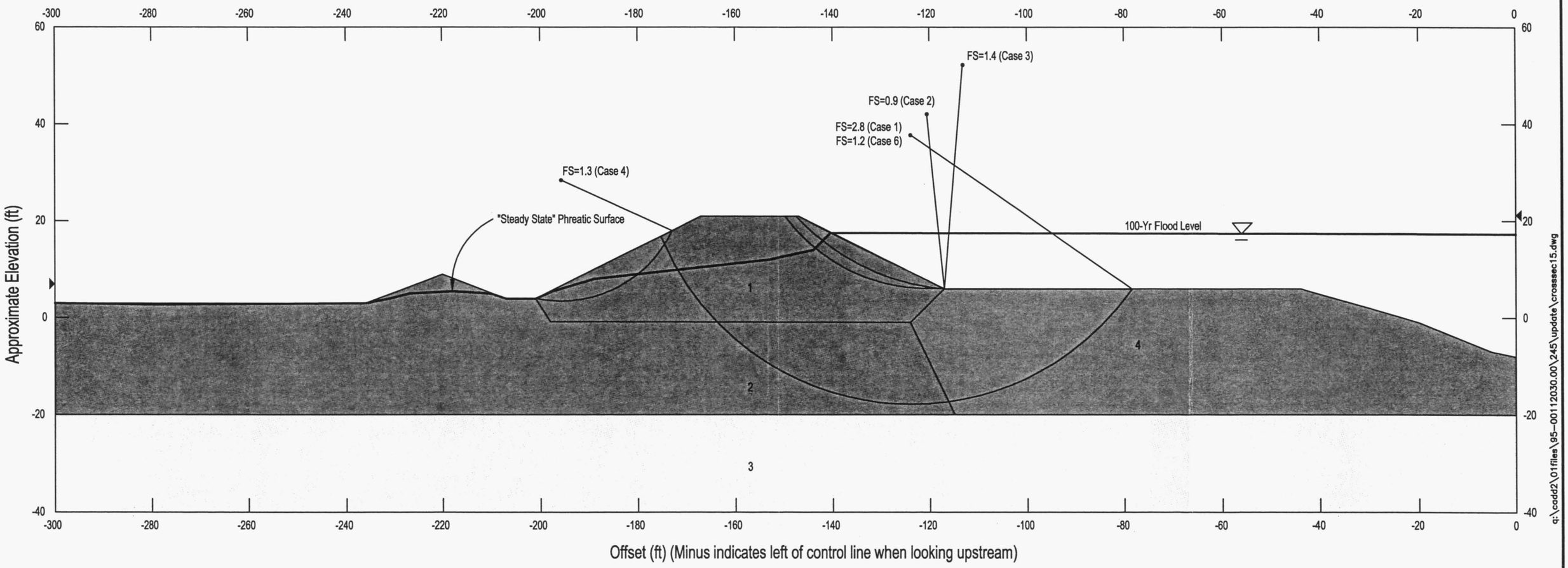
TEMP002

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		WB-03				31	290		
		Figure 22	110	20	132			2000	
1	Lean CLAY (FILL)		110	20	132	33	0	1320	Selected
		Table 6				30	150		
		WB-03				27	590		
		GRIV-16				27	0		
		Figure 22	100	30	130			800	
2	Lean CLAY		100	30	130	30	150	800	Selected
		Table 6				20	110		
		Figure 22	55	75	96			800	
3	Fat CLAY		55	75	96	20	110	800	Selected
						34	0		Estimated from N = 12, σ_v' = 2.1 ksf, Dr = 60%
4	Silty SAND				135	34	0		Selected
		Table 6				30	35		
		Figure 22	82	38	113			800	
5	Lean CLAY		82	38	113	30	35	800	Selected
6	Lean CLAY (Channel Sediments)	CB-03, CB-04	94	26	118	30	150	775	Selected
7	Fat CLAY (Channel Sediments)	CB-03, CB-04	94	26	118	20	110	750	Selected



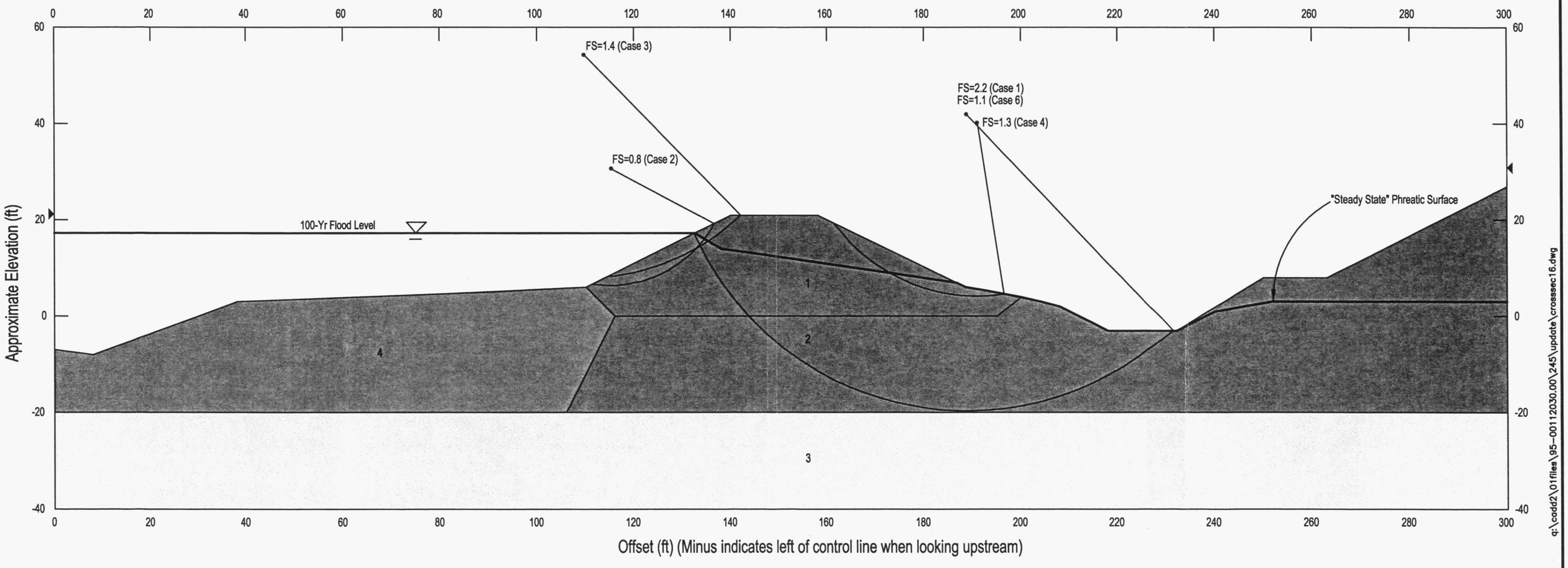
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 14 STATION 8+280 WEST LEVEE	Figure 5
TEMPO04 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		EB-09				23	750		
		Figure 23	110	21	133			2000	
1	Lean CLAY (FILL)		110	21	133	33	0	1420	Selected
		Table 6				30	35		
		EB-09				18	640		
		Figure 23	98	33	130			1000	
2	Lean CLAY		98	33	130	30	35	1000	Selected
						33	0		Estimated from N = 10,
3	Silty SAND				135	33	0		σ_v' = 1.9 ksf, Dr = 50% Selected
4	Lean CLAY (Channel Sediments)	CB-03, CB-04	94	26	118	30	35	775	Selected



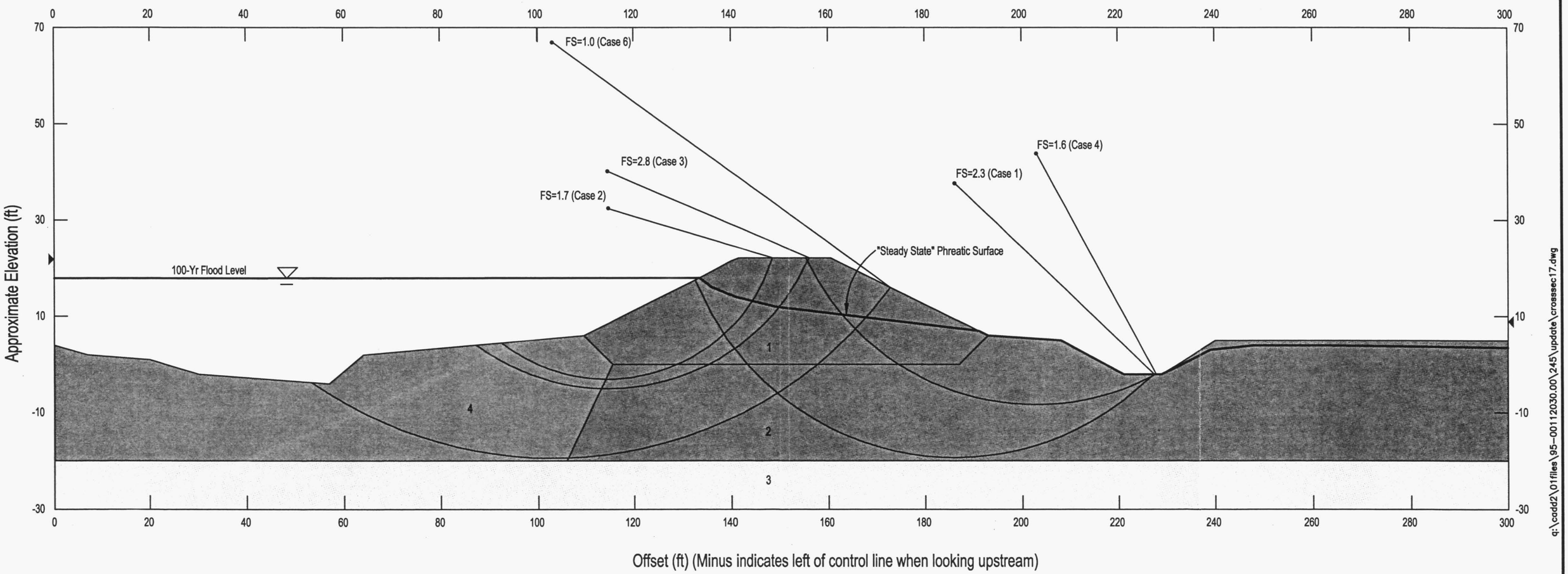
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 15 STATION 8+580 EAST LEVEE	Figure 6
7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		WB-04				36	170		
		Figure 24	110	19	131			2000	
1	Lean CLAY (FILL)		110	19	131	33	0	1420	Selected
		Table 6				30	150		
		WB-04				25	350		
		Figure 24	100	30	130			1000	
2	Lean CLAY		100	30	130	30	150	1000	Selected
						35	0		Estimated from N = 30, σ_v' = 2.7 ksf, Dr = 80%
3	Silty SAND				135	35	0		Selected
4	Lean CLAY (Channel Sediments)	CB-03, CB-04	94	26	118	30	150	775	Selected



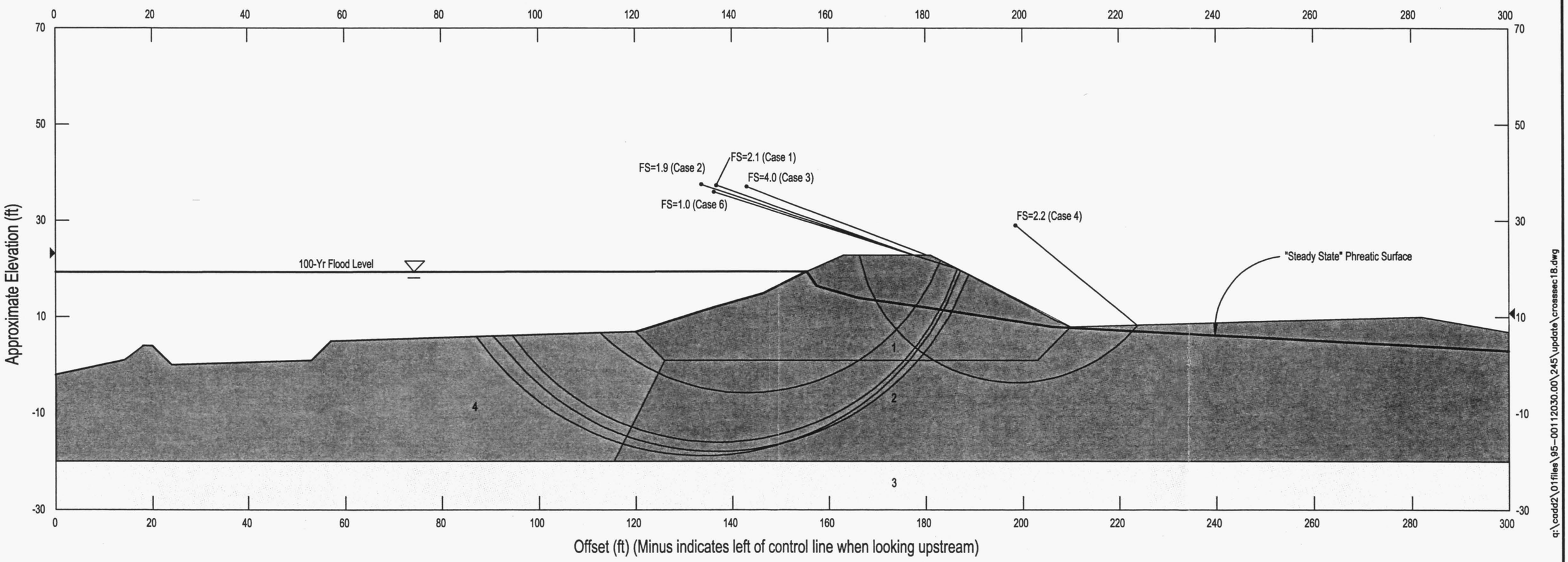
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 16 STATION 8+640 WEST LEVEE	Figure 7
TEMP004 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		WB-04				36	170		
		Figure 25	107	19	127			2000	
1	Lean CLAY (FILL)		107	19	127	24	450	1420	Selected
		Table 6				30	35		
		WB-04				25	350		
		GRIV-11				33	0		
		Figure 25	95	30	124			1000	
2	Lean CLAY		95	30	124	30	35	1000	Selected
						33	0		Estimated from N = 10,
3	Clayey SAND				135	33	0		σ_v' = 2.2 ksf, Dr = 60% Selected
4	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	35	900	Selected



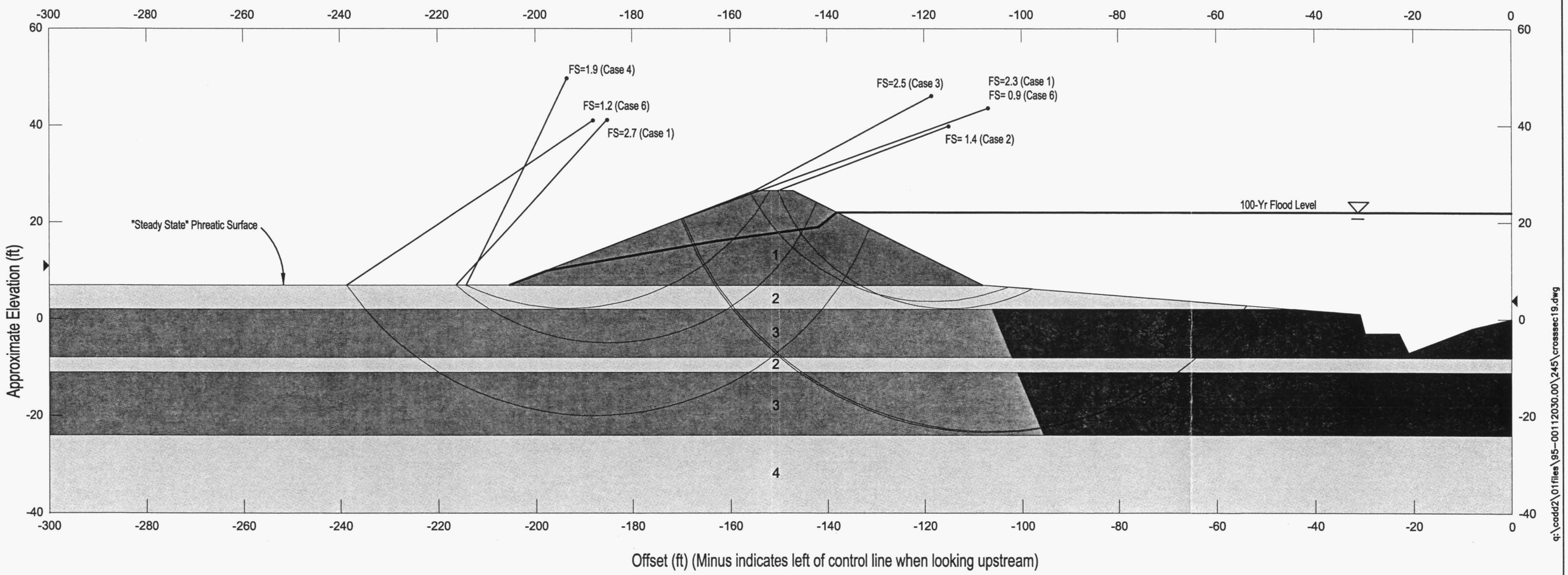
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 17 STATION 9+180 WEST LEVEE	Figure 8
TEMPOO4 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		WB-05				29	290		
		Figure 26	107	20	128			1500	
1	Lean CLAY (FILL)		107	20	128	24	450	1020	Selected
		Table 6				30	35		
		GRIV-7				35	0		
		Figure 26	95	30	124			700	
2	Lean CLAY		95	30	124	30	35	700	Selected
						33	0		Estimated from N = 10, σ_v' = 2.4 ksf, Dr = 50%
3	SAND				135	33	0		Selected
4	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	35	700	Selected



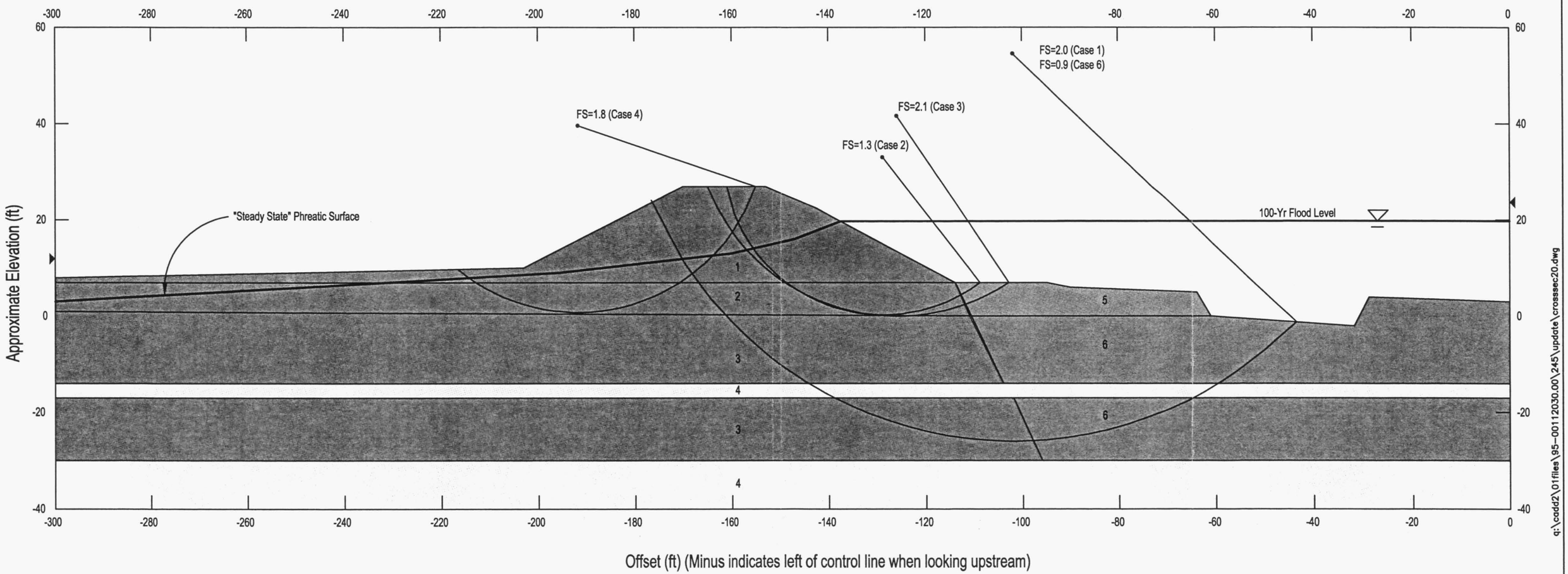
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 18 STATION 9+600 WEST LEVEE
TEMPO 7/15/02	URS	Figure 9

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		EB-09				23	750		
		Figure 27	103	20	124			2500	
1	Lean CLAY (FILL)		103	20	124	24	450	1130	Selected
						32	0		Estimated from N = 5,
									σ_v' = 1.0 ksf, Dr = 50%
2	Silty SAND				135	32	0		Selected
		Table 6				30	150		
		EB-09				18	630		
		Figure 27	102	22	124			500	
3	Lean CLAY		102	22	124	30	150	1000	Selected
						34	0		Estimated from N = 20,
									σ_v' = 2.5 ksf, Dr = 70%
4	Silty SAND				135	34	0		Selected
5	Lean CLAY (Channel Sediments)		93	30	121	30	150	700	Selected



Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 19 STATION 9+780 EAST LEVEE	Figure 10
TEMPO04 7/15/02	URS		

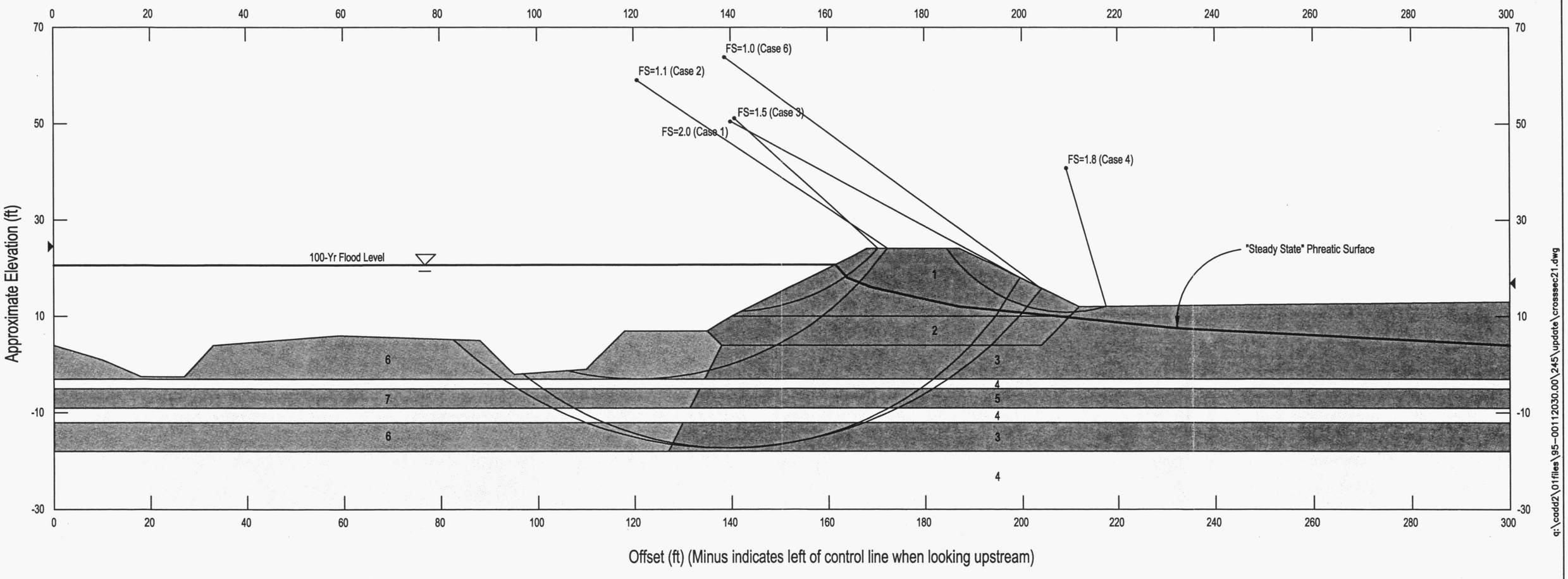
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	φ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 28	103 103	20 20	124 124	24 24	450 450	2500 1130	Selected
2	Fat CLAY	Table 6 Figure 28	90 90	33 33	120 120	20 20	110 110	500 500	Selected
3	Lean CLAY	Table 6 GRIV-7 Figure 28	100 100	30 30	130 130	30 35 30	150 0 150	750 750	Selected
4	Clayey SAND				135	34 34	0 0		Estimated from N = 20, σ_v' = 1.9 ksf, Dr = 70% Selected
5	Fat CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	20	110	500	Selected
6	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	150	750	Selected



Project No. 95-00112030.00 TEMP004 7/15/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California URS	STABILITY ANALYSIS CROSS SECTION 20 STATION 10+080 EAST LEVEE

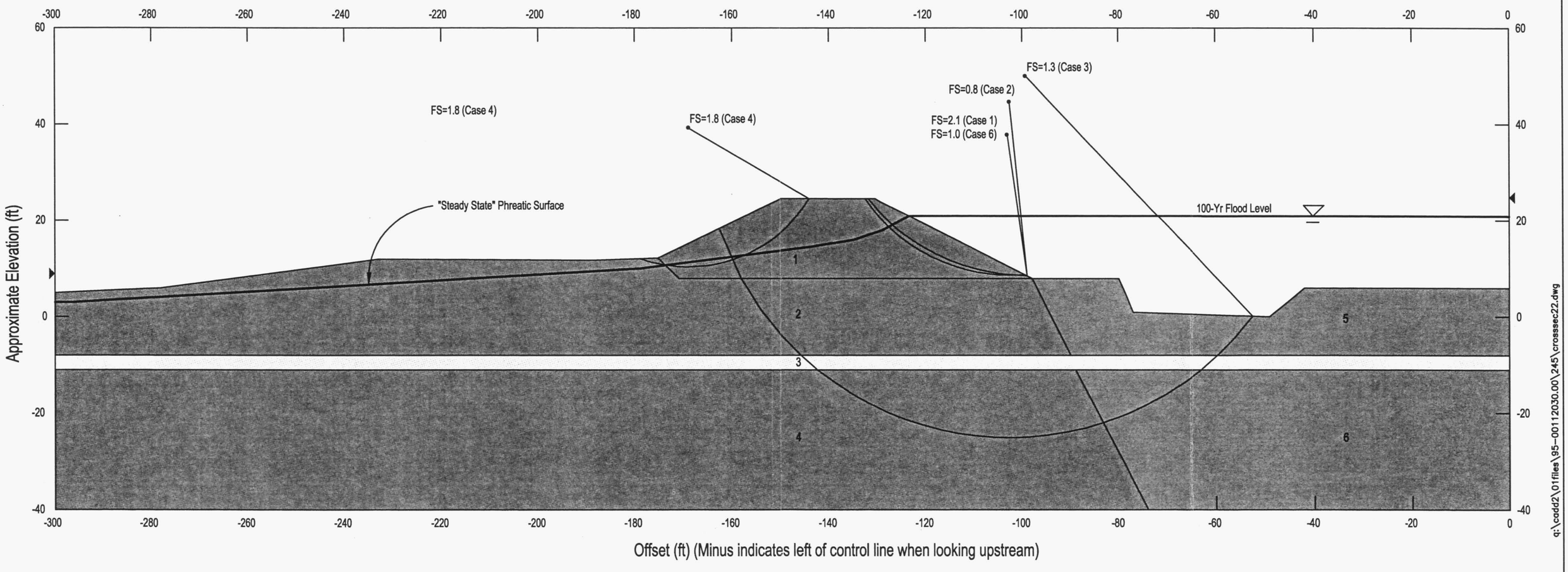
Figure 11

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 29	110 110	20 20	132 132	33 33	0 0	1800 1310	Selected
2	Lean CLAY (FILL)	Table 6 Figure 29	98 98	22 22	120 120	24 24	450 450	1000 800	Selected
3	Fat CLAY	Table 6 Figure 29	90 90	40 40	126 126	20 20	110 110	800 800	Selected
4	Clayey SAND				135	34 34	0 0		Estimated from N = 13, σ_v' = 1.6 ksf, Dr = 70% Selected
5	Lean CLAY	Table 6 Figure 29	100 100	30 30	130 130	30 30	150 150	1000 1000	Selected
6	Fat CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	20	110	800	Selected
7	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	150	900	Selected



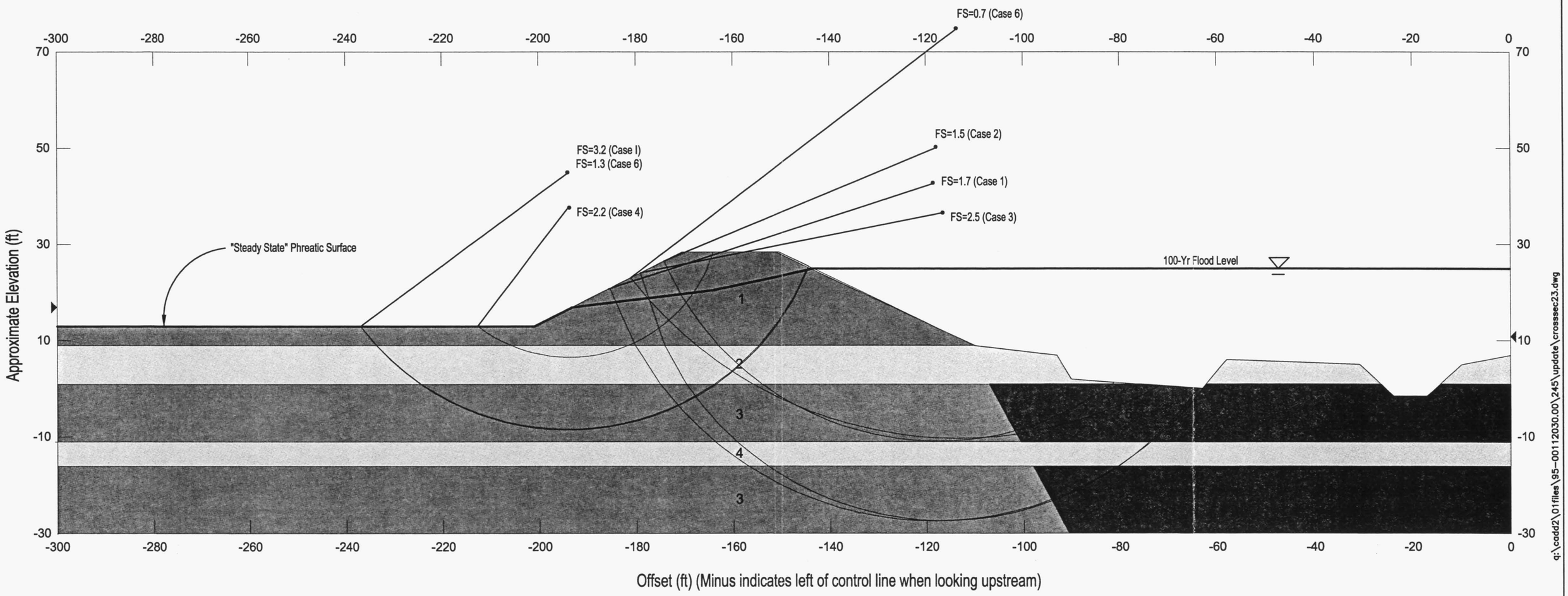
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 21 STATION 10+470 WEST LEVEE	Figure 12
TEMPO04 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 30	110 110	20 20	132 132	33 33	0 0	2000 1420	Selected
2	Lean CLAY	Table 6 Figure 30	97 97	30 30	126 126	30 30	35 35	1000 1000	Selected
3	Silty SAND				135	35 35	0 0		Estimated from N = 27, σ_v' = 2.0 ksf, Dr = 80% Selected
4	Lean CLAY	Table 6 Figure 30	105 105	22 22	128 128	30 30	150 150	1000 1000	Selected
5	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	35	1000	Selected
6	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	150	1000	Selected



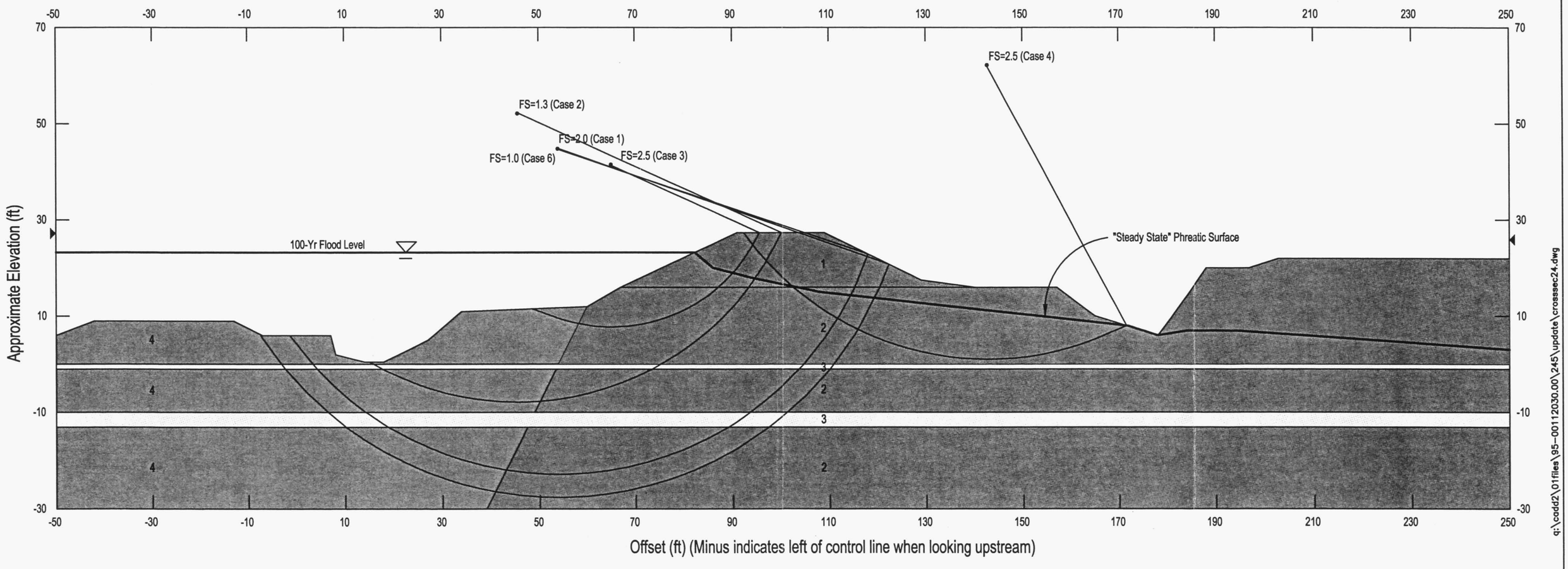
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 22 STATION 10+740 EAST LEVEE	Figure 13
TEMPO04 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		EB-11				31	140		
		Figure 31	98	20	118			2000	
1	Lean CLAY (FILL)		98	20	118	24	450	1420	Selected
									Estimated from N = 10,
						33	0		σ_v' = 1.6 ksf, Dr = 60%
2	Silty SAND				135	33	0		Selected
		Table 6				28	100		
		Figure 31	98	25	123			1000	
3	Lean CLAY		98	25	123	28	100	750	Selected
									Estimated from N = 7,
						33	0		σ_v' = 4.3 ksf, Dr = 40%
4	Silty SAND				135	33	0		Selected
5	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	35	500	Selected



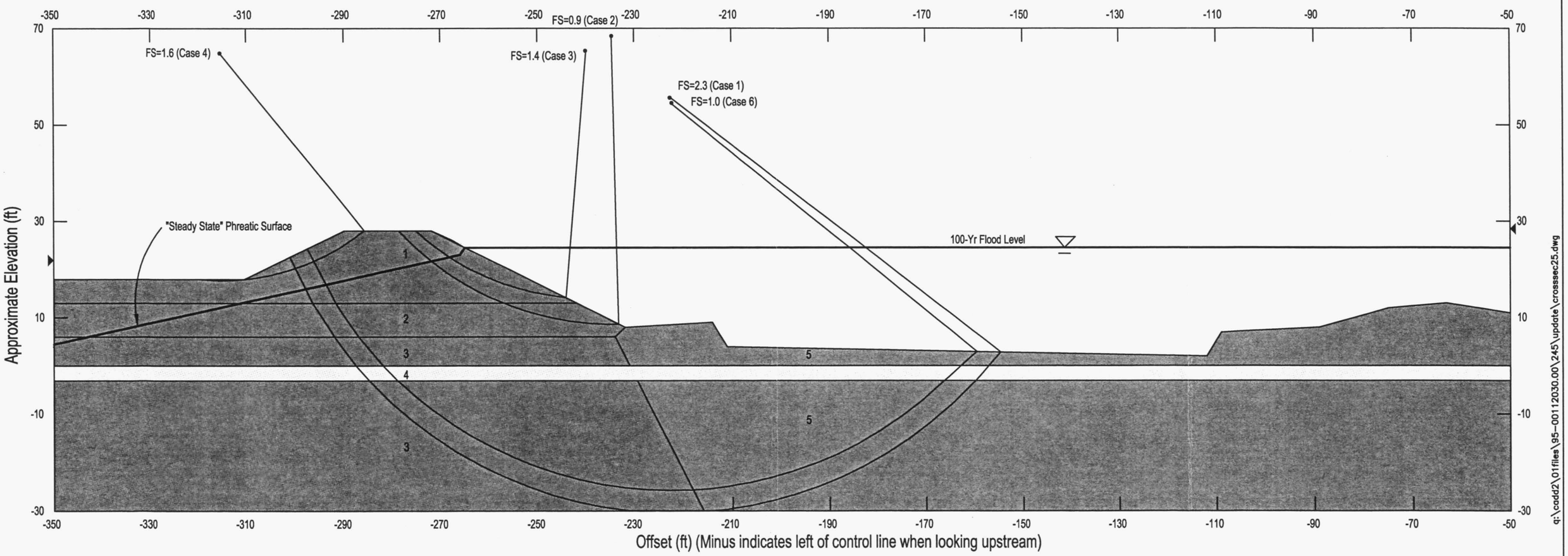
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 23 STATION 11+250 EAST LEVEE	Figure 14
TDMF004 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		WB-08				19	480		
		Figure 32	107	21	129			2000	
1	Lean CLAY (FILL)		107	21	129	24	450	1420	Selected
		Table 6				30	35		
		WB-08				25	170		
		Figure 32	95	32	125			1000	
2	Lean CLAY		95	32	125	30	35	1000	Selected
						35	0		Estimated from N = 19, σ_v' = 1.5 ksf, Dr = 80%
3	Silty SAND				135	35	0		Selected
4	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	35	500	Selected



Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 24 STATION 11+790 WEST LEVEE	Figure 15
TEMP004 7/15/02	URS		

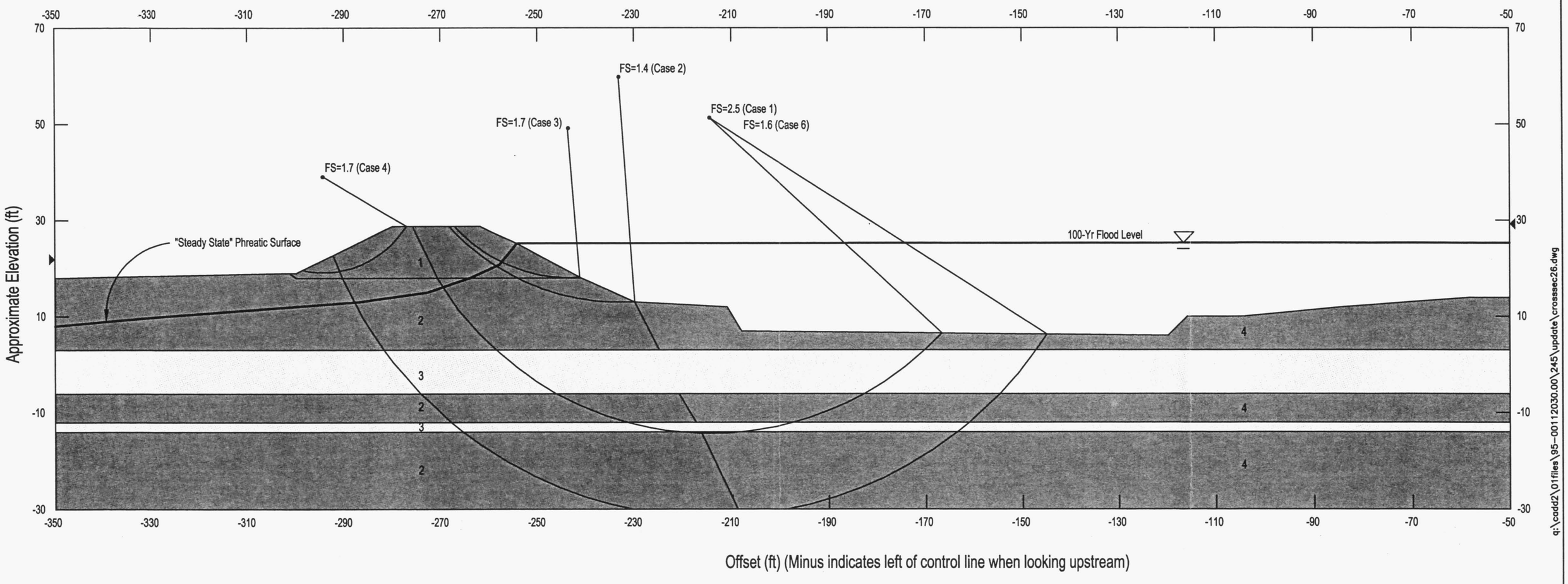
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 33	110 110	19 19	131 131	33 33	0 0	2000 1540	Selected
2	Fat CLAY	Table 6 Figure 33	100 100	28 28	128 128	20 20	110 110	1500 1500	Selected
3	Lean CLAY	Table 6 Figure 33	107 107	20 20	128 128	30 30	150 150	1500 1500	Selected
4	Silty SAND				135	34 34	0 0		Estimated from N = 20, σ_v' = 1.9 ksf, Dr = 70% Selected
5	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected



Project No. 95-00112030.00 TEMP004 7/15/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California URS	STABILITY ANALYSIS CROSS SECTION 25 STATION 12+120 EAST LEVEE

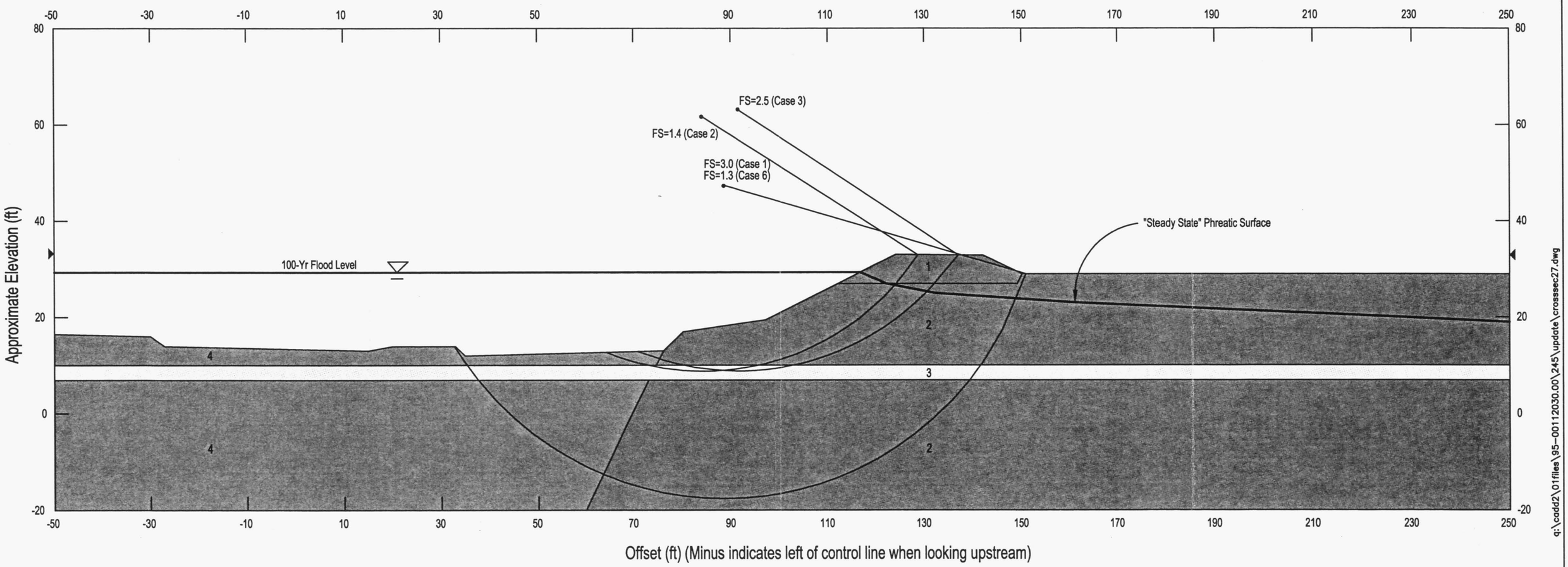
Figure 16

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		EB-14				36	320		
		Figure 34	110	18	130			2000	
1	Lean CLAY (FILL)		110	18	130	33	0	1460	Selected
		Table 6				30	150		
		Figure 34	100	25	125			1200	
2	Lean CLAY		100	25	125	30	150	1200	Selected
						35	0		Estimated from N = 35, σ_v' = 2.2 ksf, Dr = 90%
3	Silty SAND				135	35	0		Selected
4	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected



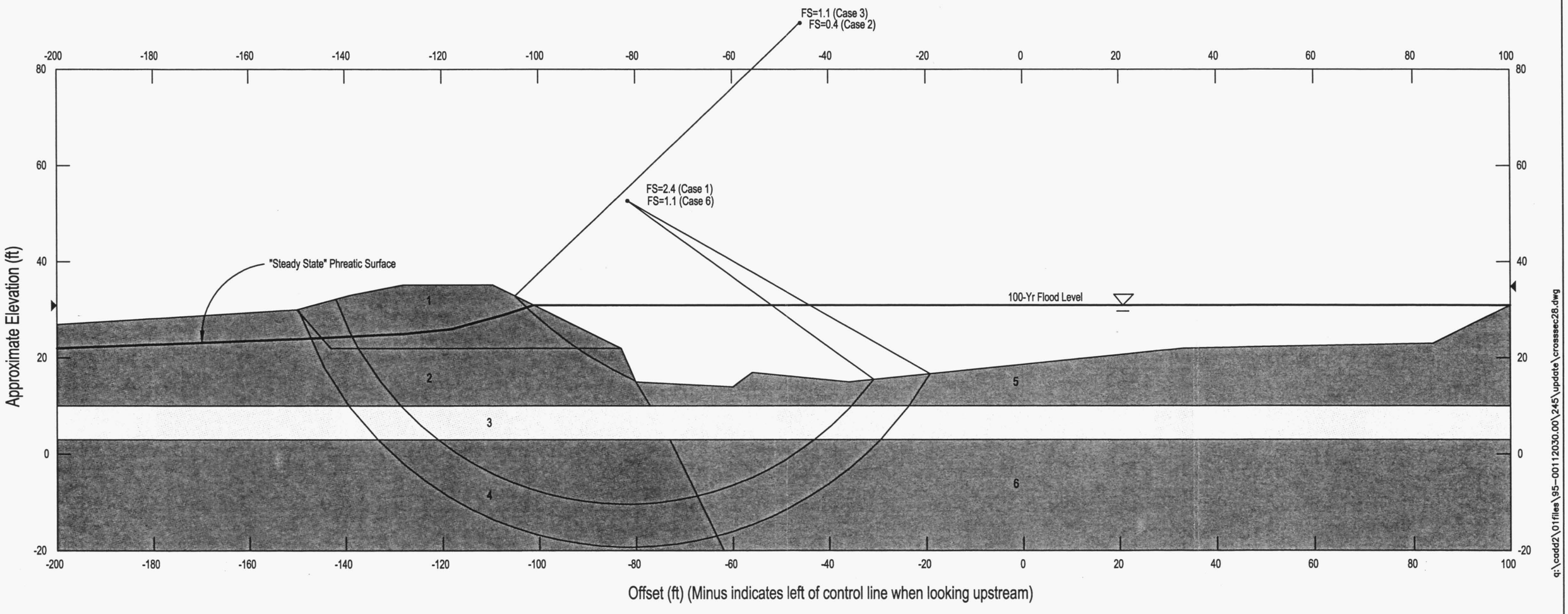
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 26 STATION 12+810 EAST LEVEE	Figure 17
TEMP004 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 35	116 116	19 19	138 138	33 33	0 0	3000 2040	Selected
2	Lean CLAY	Table 6 Figure 35	100 100	24 24	124 124	30 30	150 150	1400 1400	Selected
3	SAND				135	35 35	0 0		Estimated from N = 25, σ_v' = 3.1 ksf, Dr = 70% Selected
4	Lean CLAY (Channel Sediments)	Table 6	97	27	123 123	30 30	150 150	750	Selected



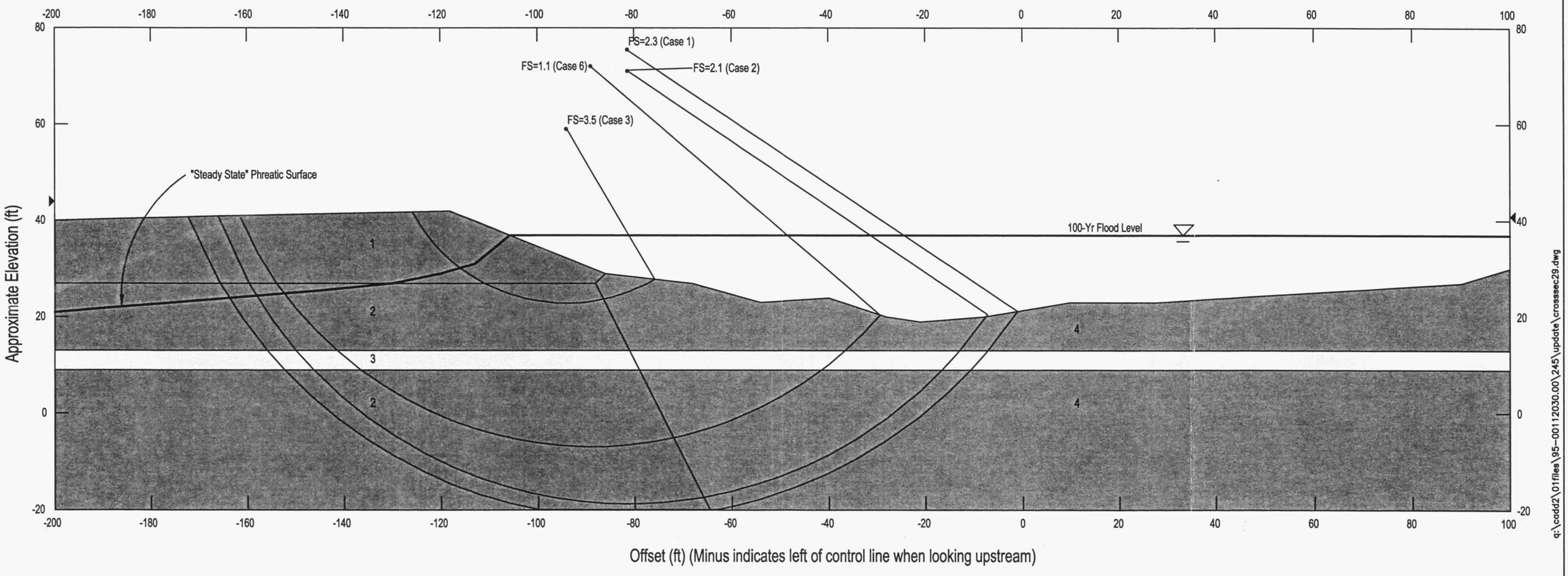
Project No. 95-00112030.00 TEMP004 7/15/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California URS	STABILITY ANALYSIS CROSS SECTION 27 STATION 13+890 WEST LEVEE	Figure 18

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 36	110 110	19 19	131 131	33 33	0 0	2000 1420	Selected
2	Lean CLAY	Table 6 Figure 36	98 98	22 22	120 120	30 30	35 35	1000 1000	Selected
3	Silty SAND				135	34 34	0 0		Estimated from N = 25, σ_v' = 1.8 ksf, Dr = 80% Selected
4	Lean CLAY	Table 6 Figure 36	105 105	22 22	128 128	30 30	150 150	1000 1000	Selected
5	Lean CLAY (Channel Sediments)	CB-09, CB-10	97	27	123	30	150	750	Selected



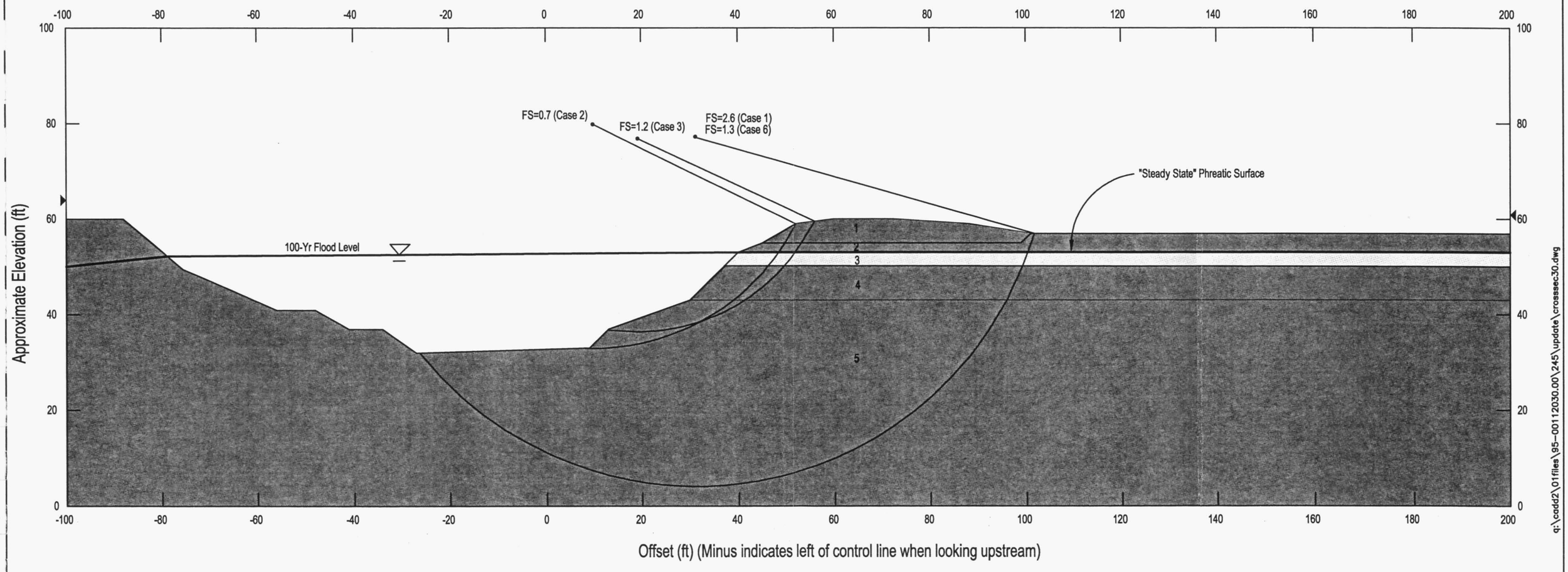
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 28 STATION 14+370 EAST LEVEE	Figure 19
TEMP004 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 37	105 105	20 20	126 126	24 24	450 450	2000 1420	Selected
2	Lean CLAY	Table 6 Figure 37	100 100	20 20	120 120	30 30	150 150	1000 1000	Selected
3	Silty SAND with gravel				135	36 36	0 0		Estimated from N = 34, σ_v' = 3.0 ksf, Dr = 80% Selected
4	Lean CLAY (Channel Sediments)	CB-09, CB-10	97	27	123	30	150	750	Selected

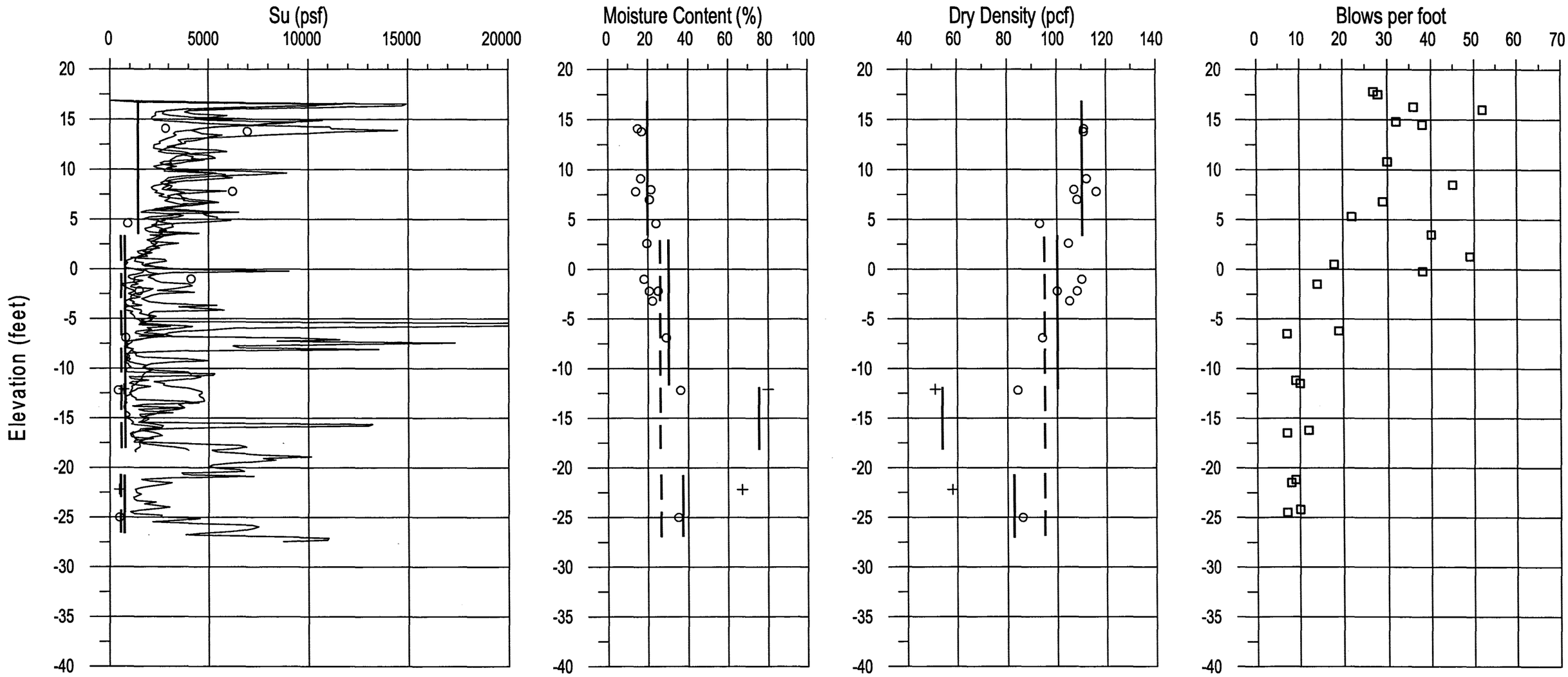


Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 29 STATION 15+176 EAST LEVEE	Figure 20
TEMPFOOM 7/15/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 38	110 110	19 19	131 131	33 33	0 0	3000 2140	Selected
2	Fat CLAY	Table 6 Figure 38	85 85	28 28	109 109	20 20	110 110	1500 1500	Selected
3	Silty SAND				135	34 34	0 0		Estimated from N = 22, σ_v' = 1.6 ksf, Dr = 80% Selected
4	Lean CLAY	Table 6 Figure 38	100 100	20 20	120 120	30 30	150 150	2000 2000	Selected
5	Lean CLAY	Table 6 WB-17 Figure 38	98 98	25 25	123 123	30 30	35 620 35	1500 1500	Selected



Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 30 STATION 17+370 WEST LEVEE	Figure 21
TEMP004 7/15/02	URS		



Data from the following borings and CPTs:

EB-08
WB-03
EC-02
EC-03
EC-04

+ Fat Clay

○ Lean Clay

□ All Soil Types

LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

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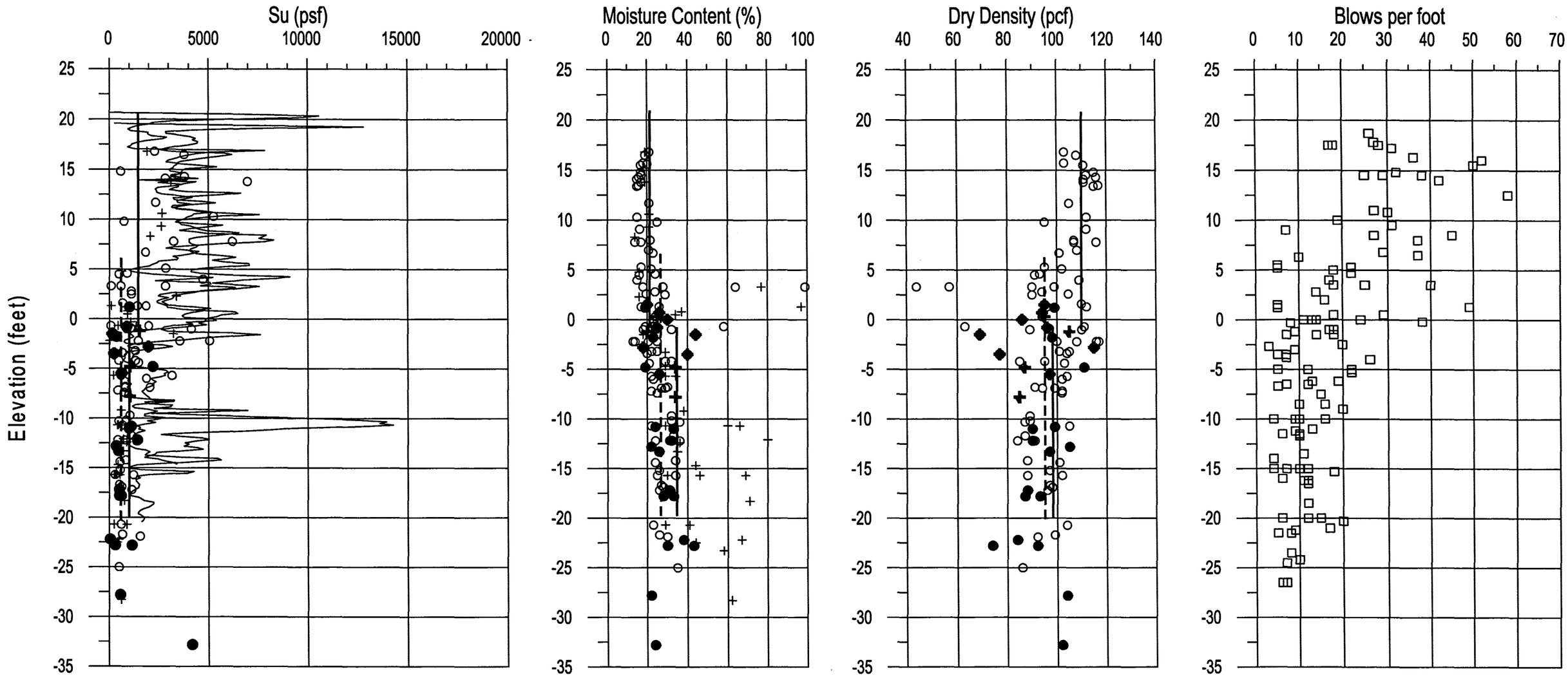
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO 14	Figure 22
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-10	OPS-1	TDB-3	EB-21	EC-05
GRIV-11	OPS-2	TDB-4	EB-22	WC-03
GRIV-13	OPS-3	TDB-5	WB-03	
GRIV-14	OPS-4	EB-07	WB-04	
GRIV-17	TDB-1	EB-08	CB-03	
GTPP-B13	TDB-2	EB-09	CB-04	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

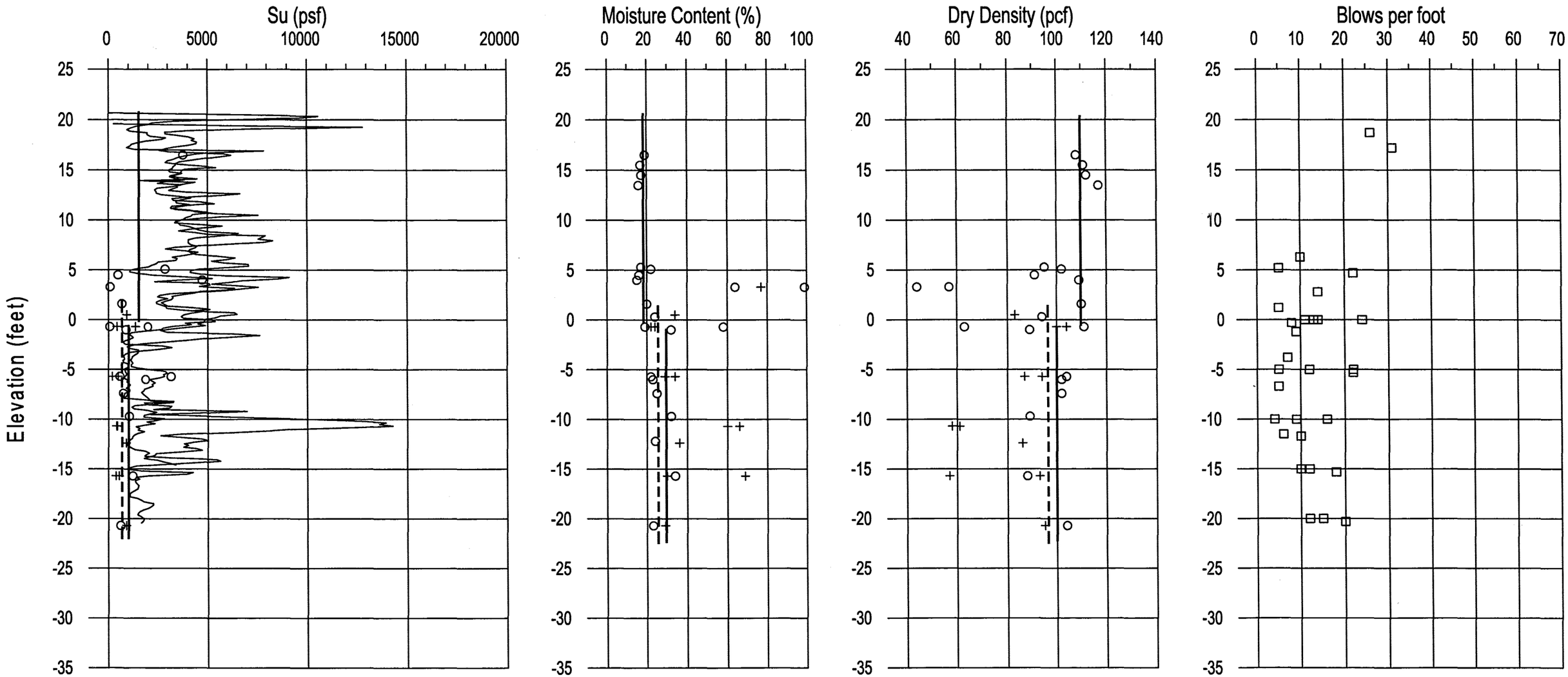
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\index15.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 15	Figure 23
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-11	OPS-1	WB-04	EC-05
GRIV-12	OPS-2	CB-03	
GRIV-13	OPS-3	CB-04	
GRIV-14	OPS-4	WC-03	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

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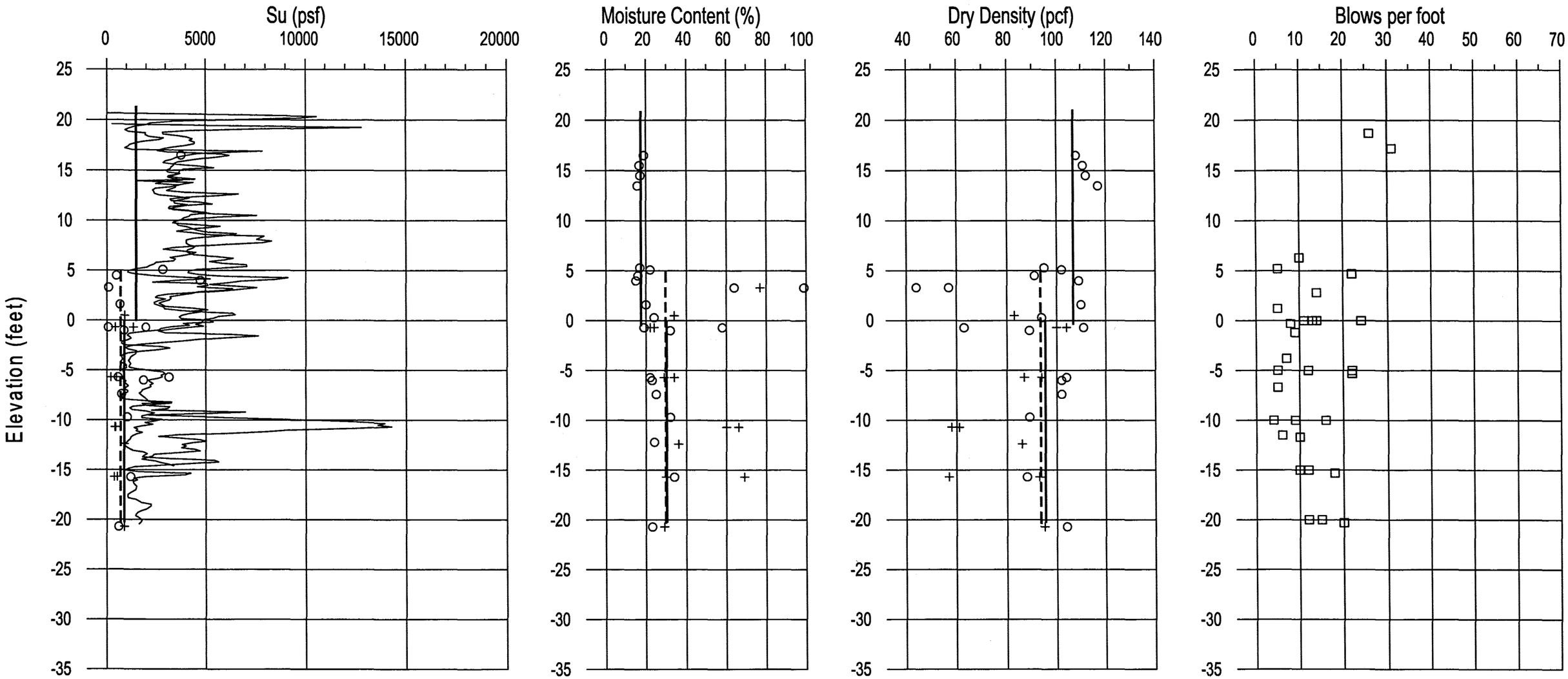
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 16	Figure 24
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-11	OPS-1	WB-04	EC-05
GRIV-12	OPS-2	CB-05	
GRIV-13	OPS-3	CB-06	
GRIV-14	OPS-4	WC-03	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

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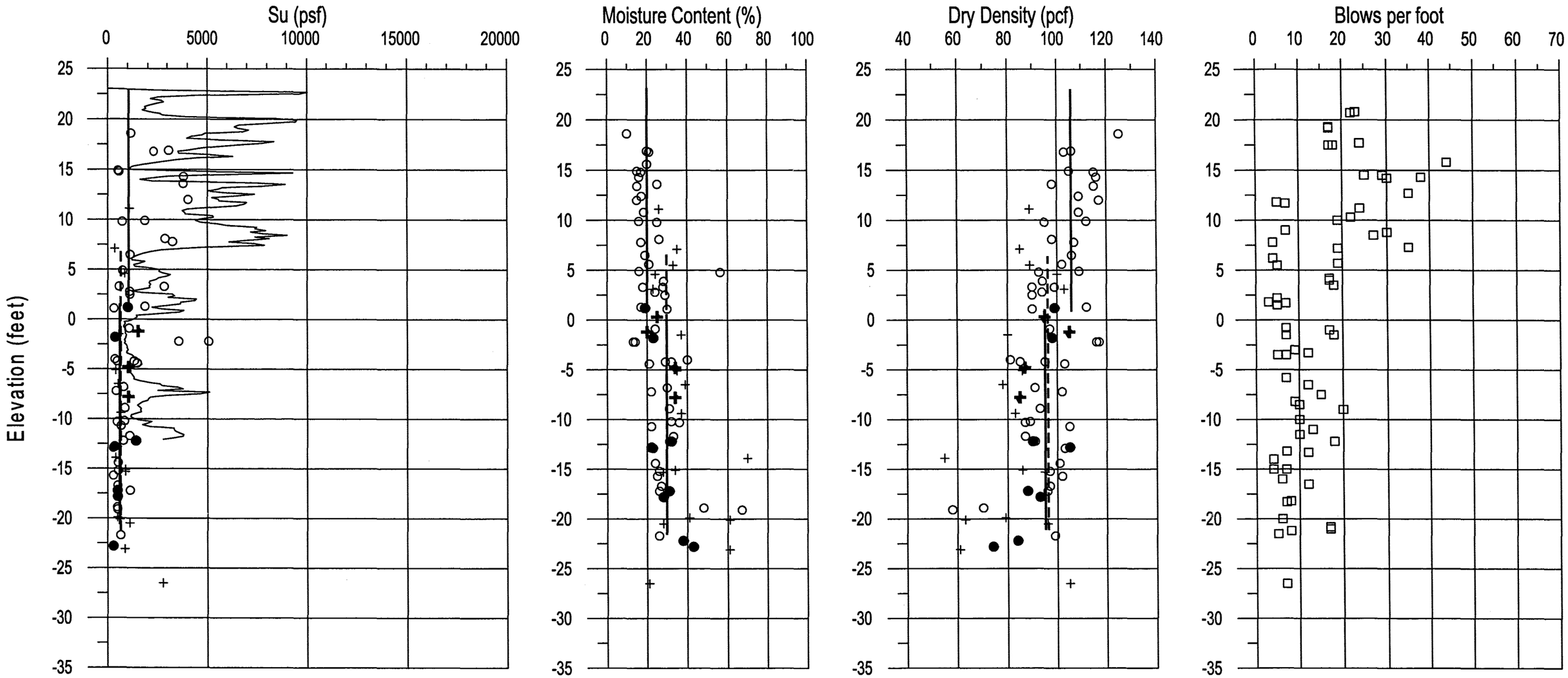
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 17	Figure 25
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-10	TDB-3	WB-21
GRIV-7	TDB-4	WB-22
GRIV-8	TDB-5	CB-05
GRIV-9	EB-09	CB-06
TDB-1	EB-23	WC-04
TDB-2	WB-05	

Levee Fill and Foundation Soil

+ Fat Clay

○ Lean Clay

□ All Soil Types

Channel Sediments

+ Fat Clay

● Lean Clay

■ All Soil Types

LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

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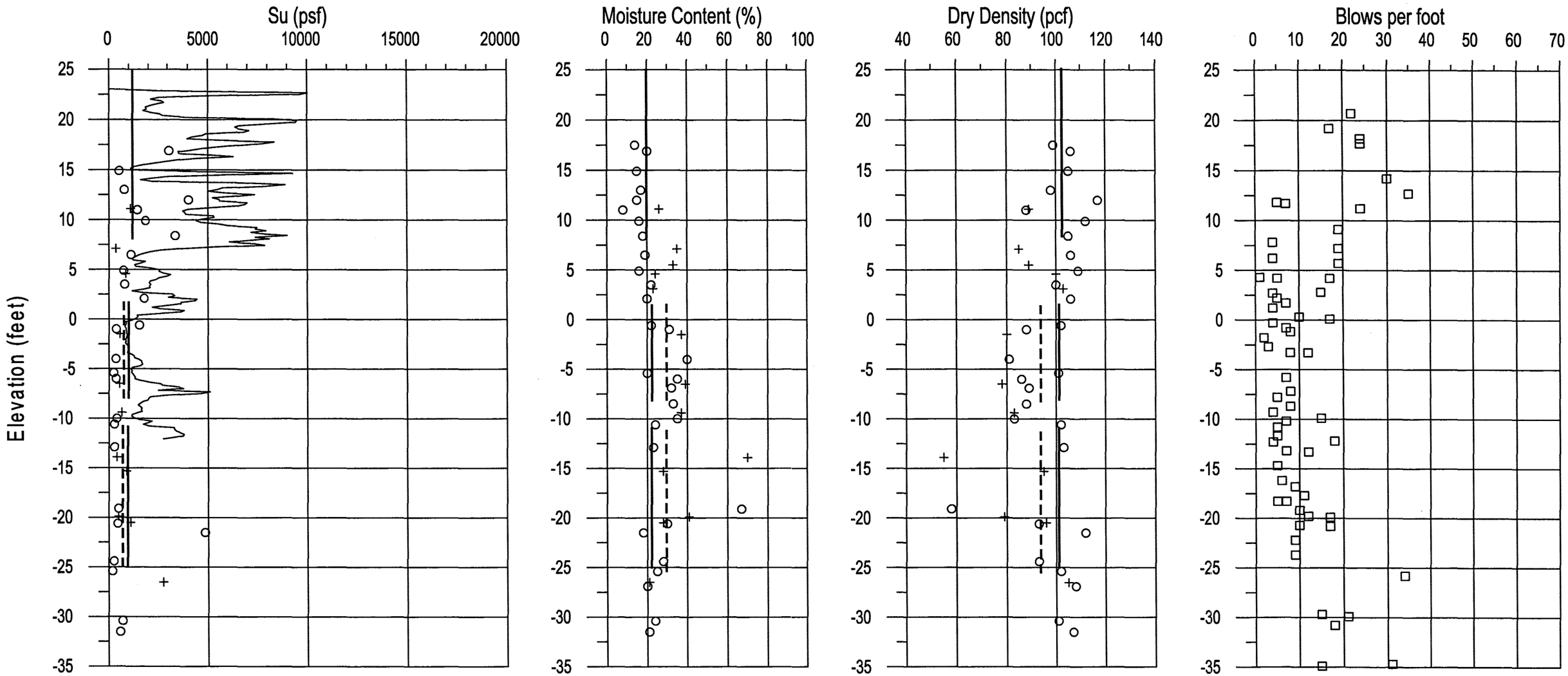
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 18	Figure 26
7/15/02			
	URS		



Data from the following borings and CPTs:

GRIV-7	WB-05
GRIV-8	WB-22
HHP-1	CB-05
HHP-3	CB-06
HHP-4	WC-04

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

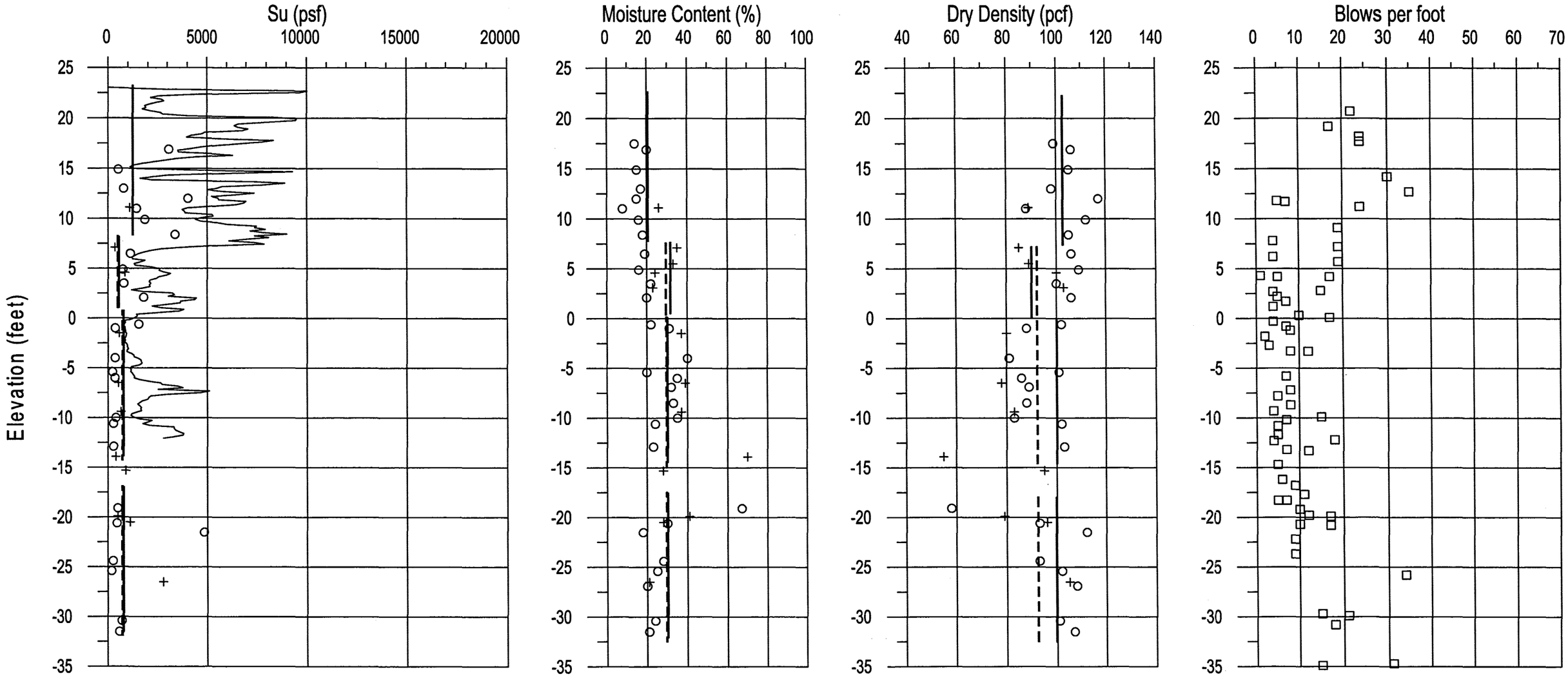
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index19.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 19	Figure 27
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-7	WB-05
GRIV-8	WB-22
HHP-1	CB-05
HHP-3	CB-06
HHP-4	WC-04

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

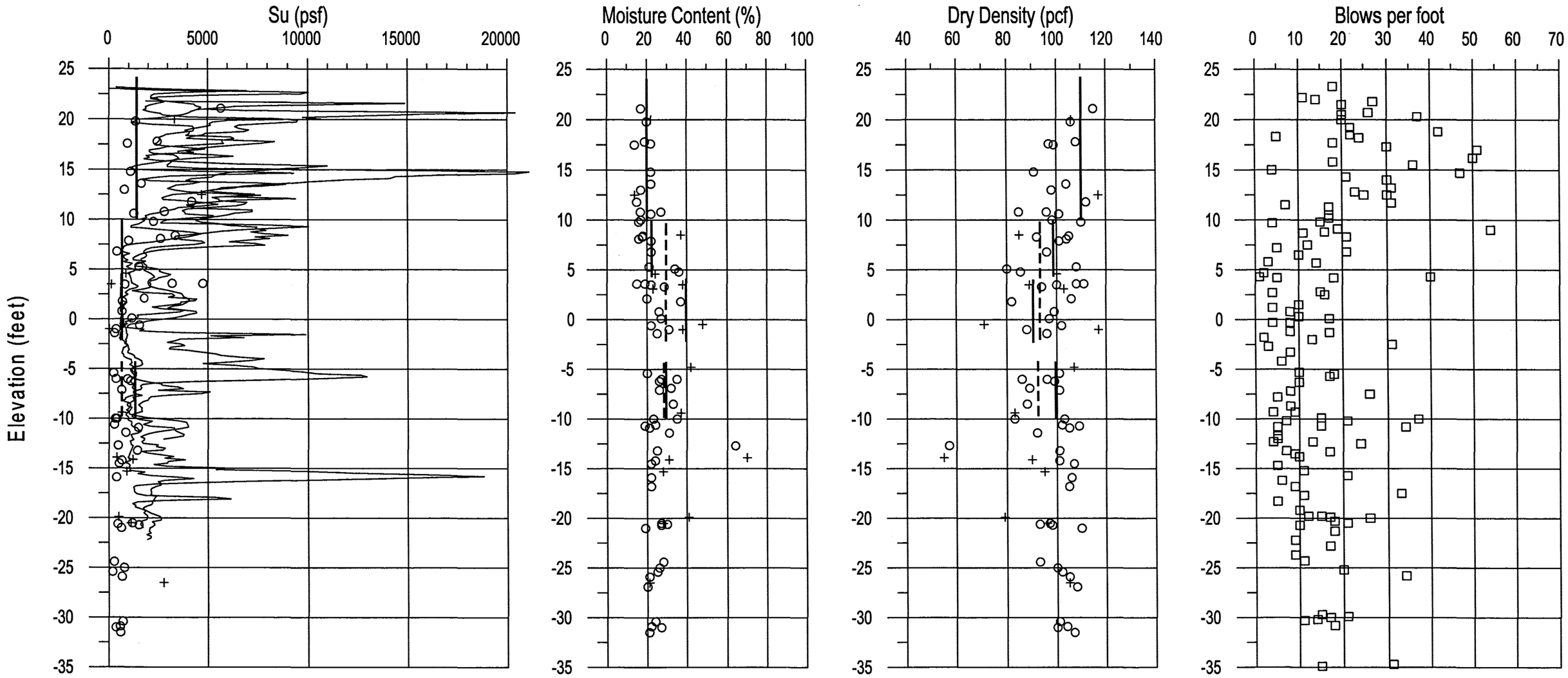
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index20.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 20	Figure 28
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-3	HHP-3	EB-10	CB-06
GRIV-4	HHP-4	EB-11	EC-06
GRIV-6	HHP-5	EB-12	WC-04
HHP-1	HHPA-2	WB-06	WC-05
HHP-2	HHPA-3	CB-05	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

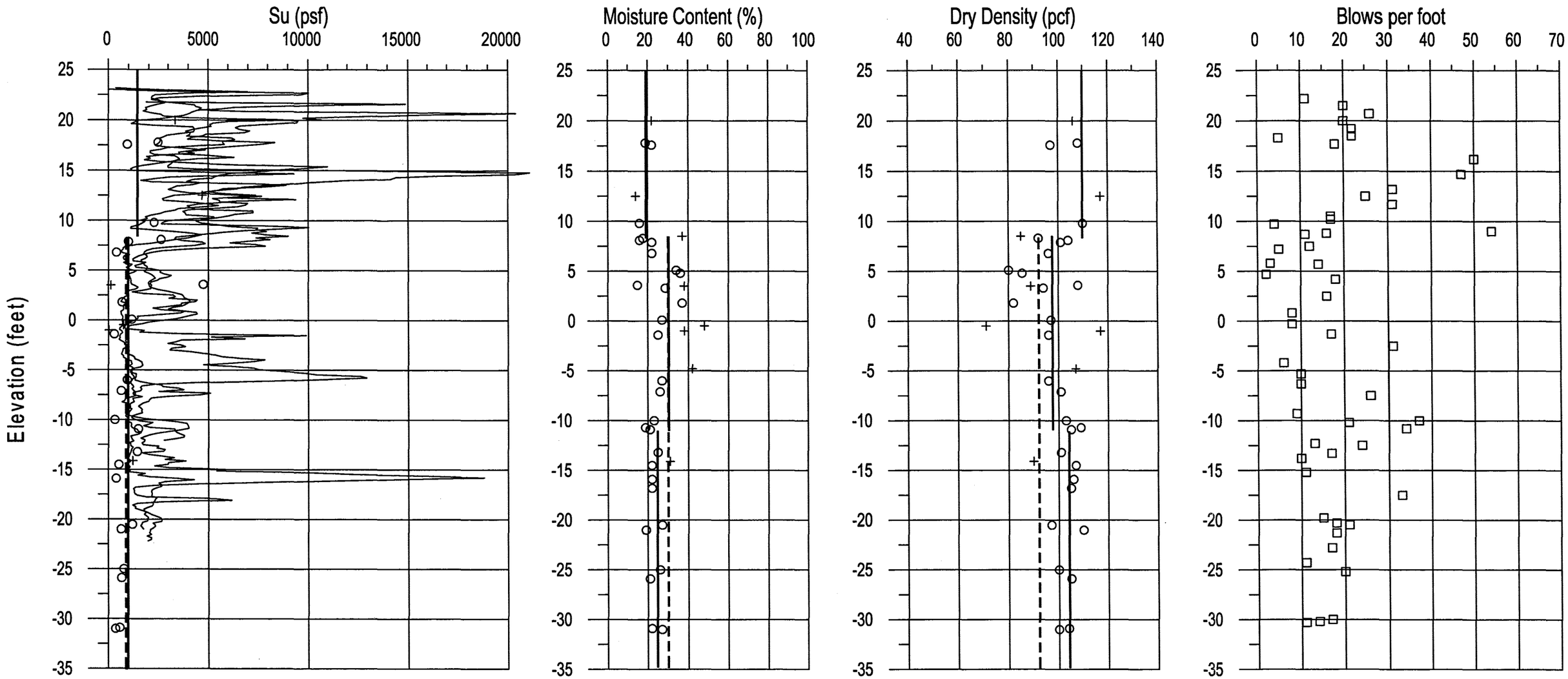
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index21.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 21	Figure 29
7/15/02			
	URS		



Data from the following borings and CPTs:

GRIV-4	HHPA-3	EC-06
GRIV-6	EB-10	WC-04
HHP-2	WB-06	WC-05
HHP-5	CB-05	
HHPA-2	CB-06	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

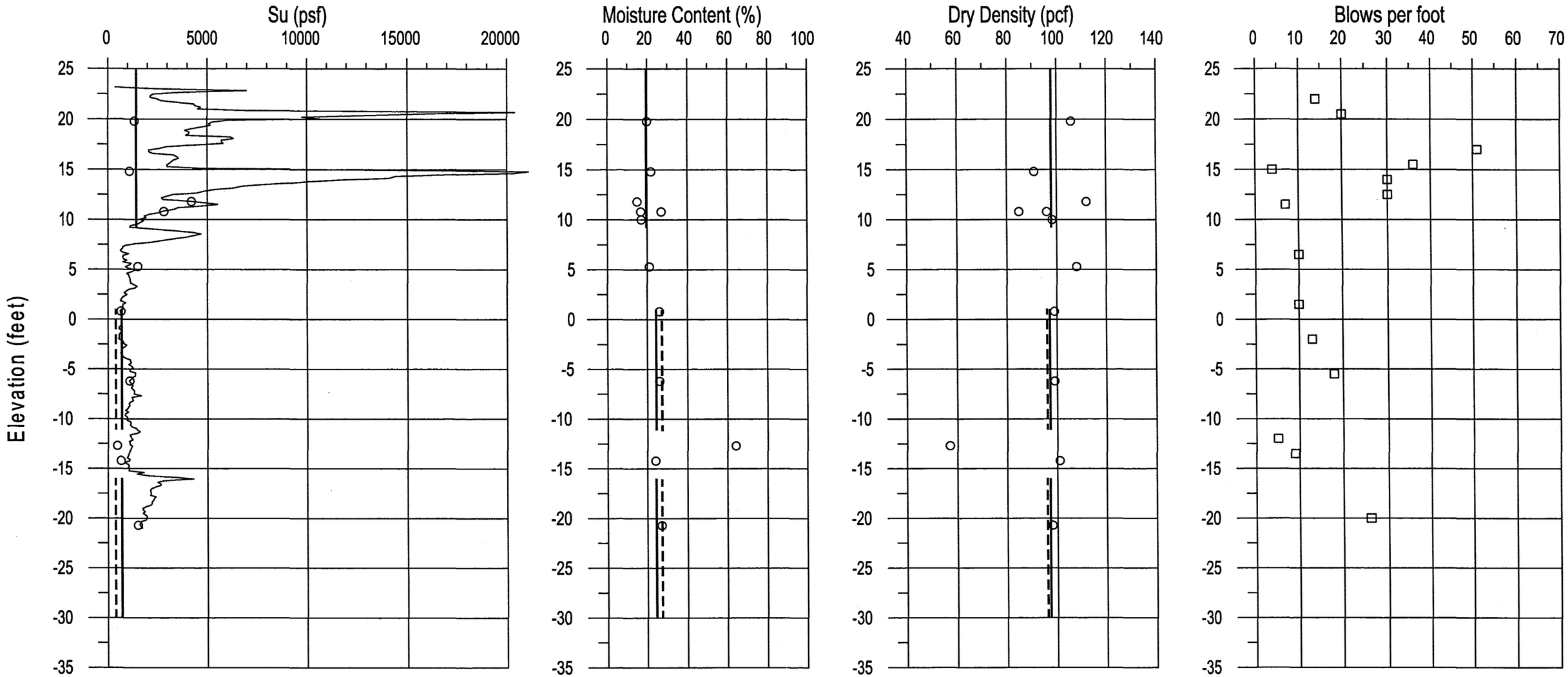
LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index22.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 22	Figure 30
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-3	CB-07
EB-11	CB-08
WC-05	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

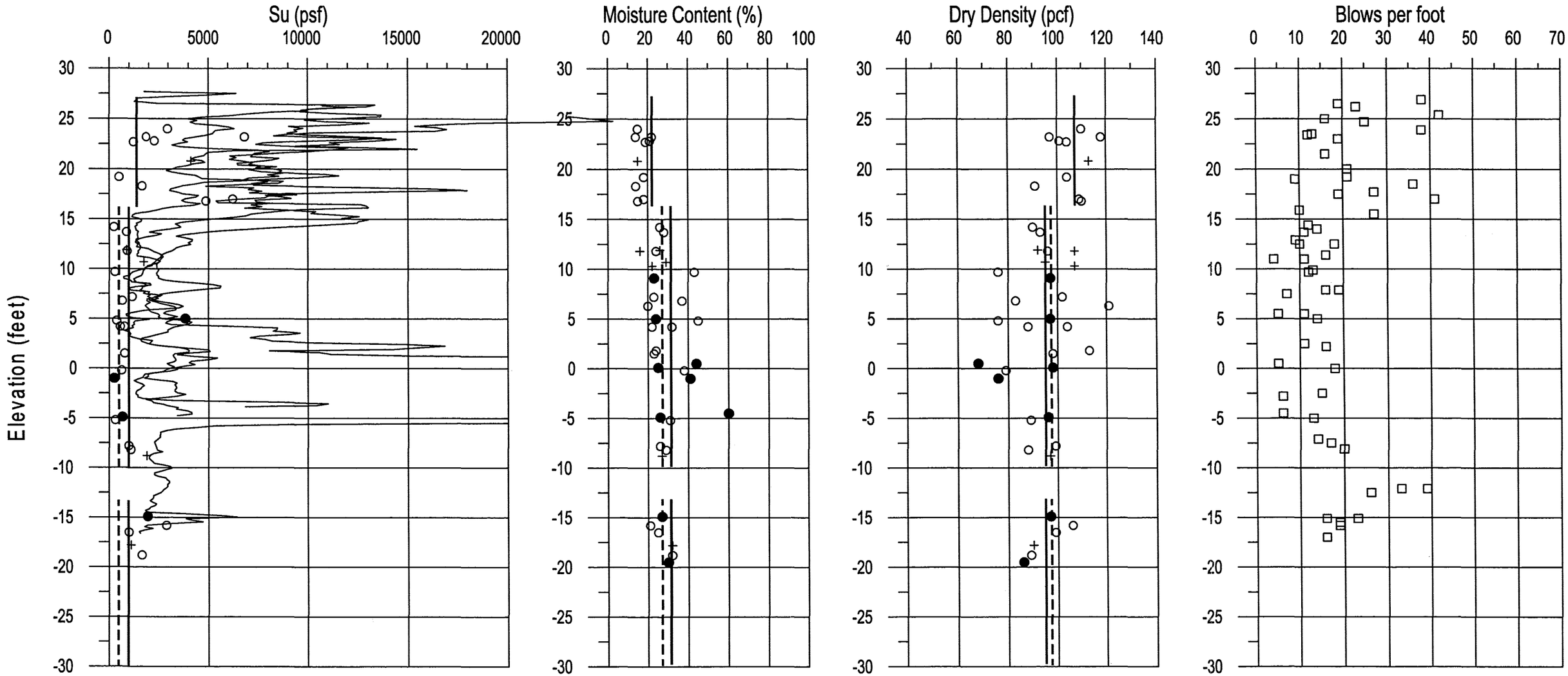
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\update\index23.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 23	Figure 31
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-1	SPRR-5	EB-13	WB-09	EC-09
SPRR-1	SPRR-7	EB-14	WB-10	EC-10
SPRR-2	SPRR-8	EB-24	WB-11	EC-11
SPRR-3	SPRR-9	WB-07	CB-07	
SPRR-4	SPRR-10	WB-08	CB-08	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

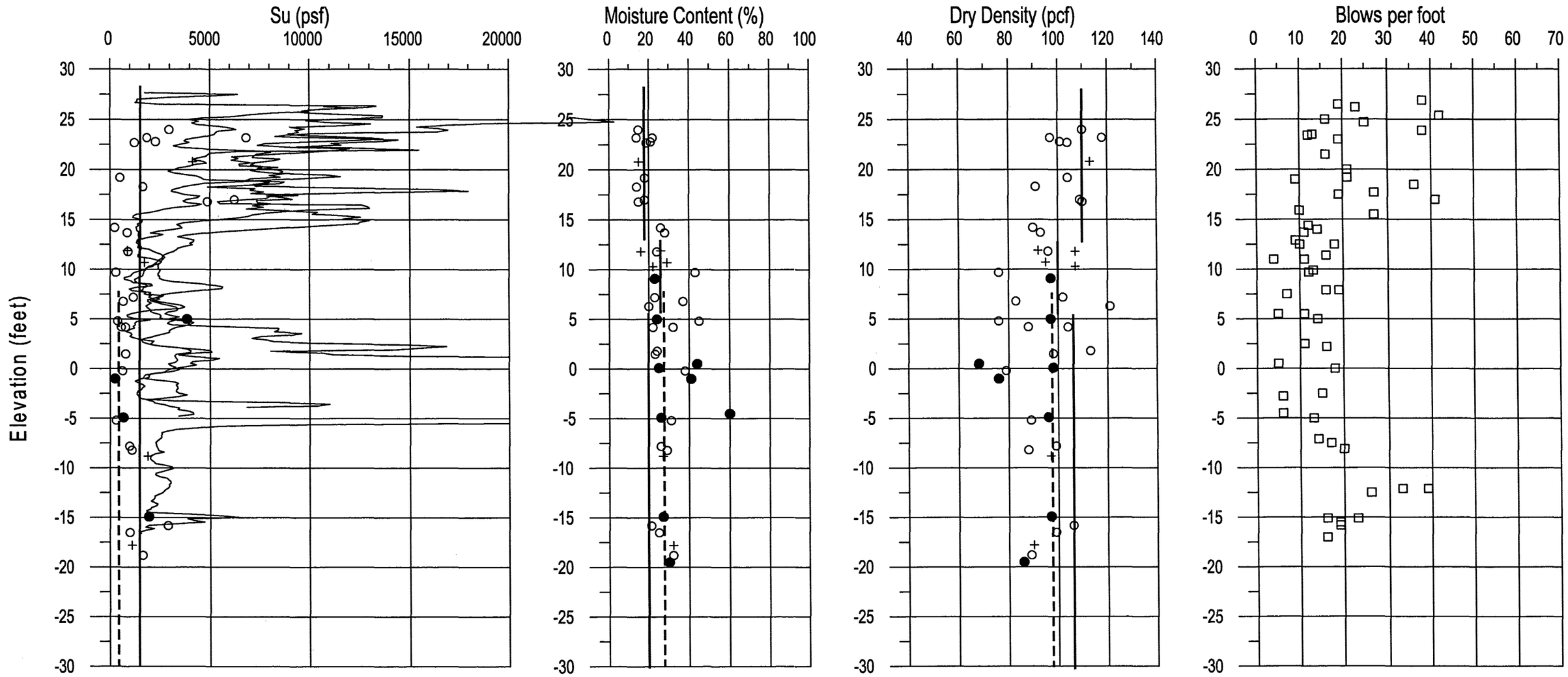
- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

LEGEND:

- SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.
- SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index24.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 24	Figure 32
7/15/02	URS		



Data from the following borings and CPTs:

GRIV-1	SPRR-5	EB-13	WB-09	EC-09
SPRR-1	SPRR-7	EB-14	WB-10	EC-10
SPRR-2	SPRR-8	EB-24	WB-11	EC-11
SPRR-3	SPRR-9	WB-07	CB-07	
SPRR-4	SPRR-10	WB-08	CB-08	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

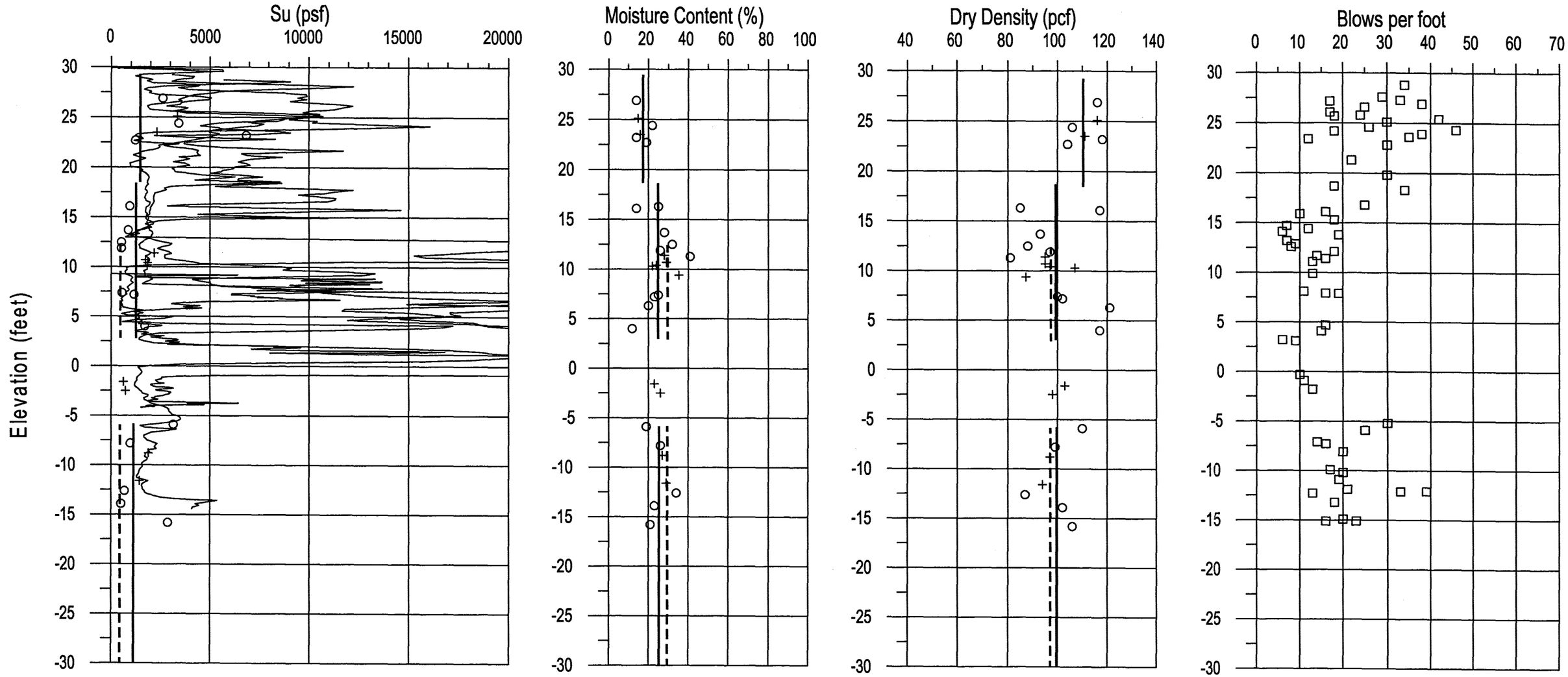
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index25.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 25	Figure 33
7/15/02	URS		



Data from the following borings and CPTs:

SPRR-3	EB-13	WB-12	EC-10
SPRR-4	EB-14	WB-13	EC-11
SPRR-5	WB-10	CB-07	EC-12
SPRR-6	WB-11	CB-08	WC-06

Levee Fill and Foundation Soil

+ Fat Clay

○ Lean Clay

□ All Soil Types

Channel Sediments

+ Fat Clay

● Lean Clay

■ All Soil Types

LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

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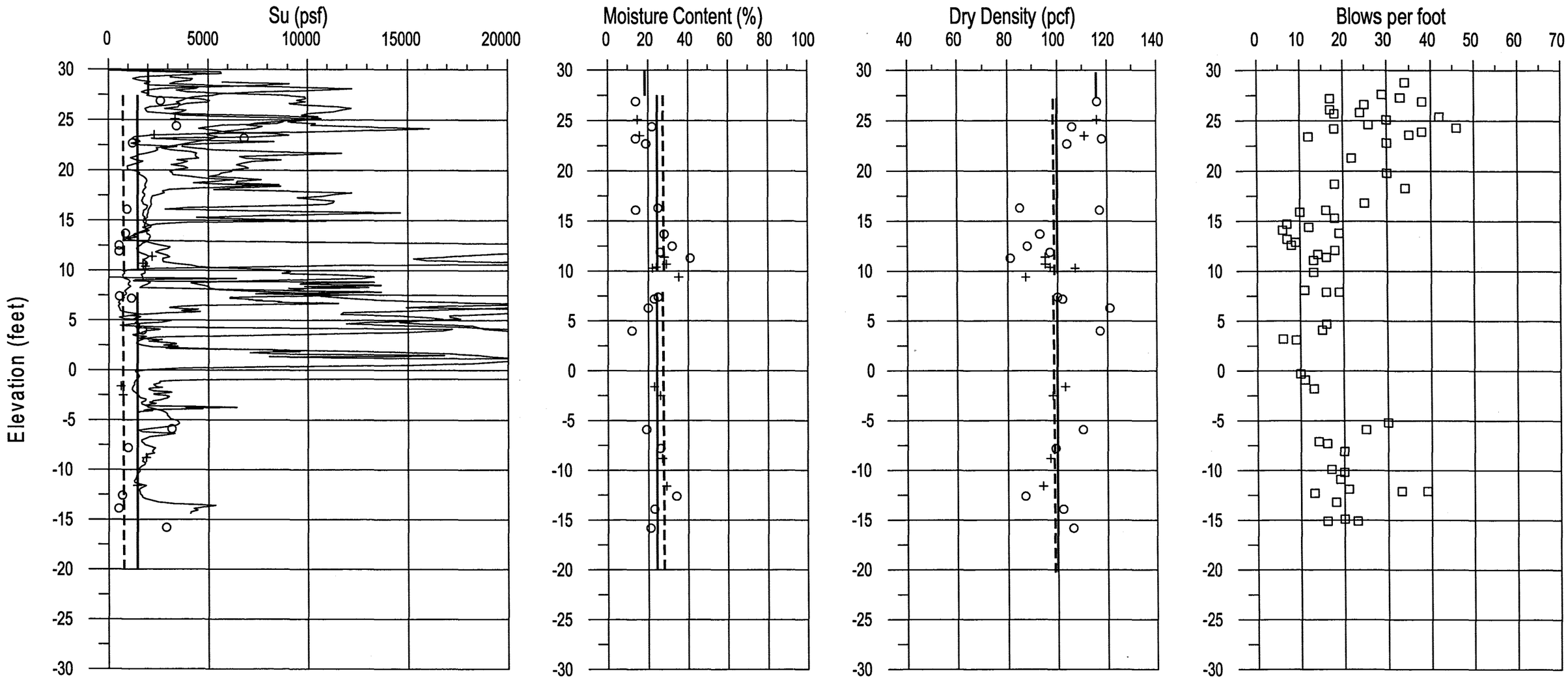
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 26	Figure 34
7/15/02	URS		



Data from the following borings and CPTs:

SPRR-3	EB-13	WB-12	EC-10	WC-07
SPRR-4	EB-14	WB-13	EC-11	WC-08
SPRR-5	WB-10	CB-07	EC-12	
SPRR-6	WB-11	CB-08	WC-06	

Levee Fill and Foundation Soil

+ Fat Clay

○ Lean Clay

□ All Soil Types

Channel Sediments

+ Fat Clay

● Lean Clay

■ All Soil Types

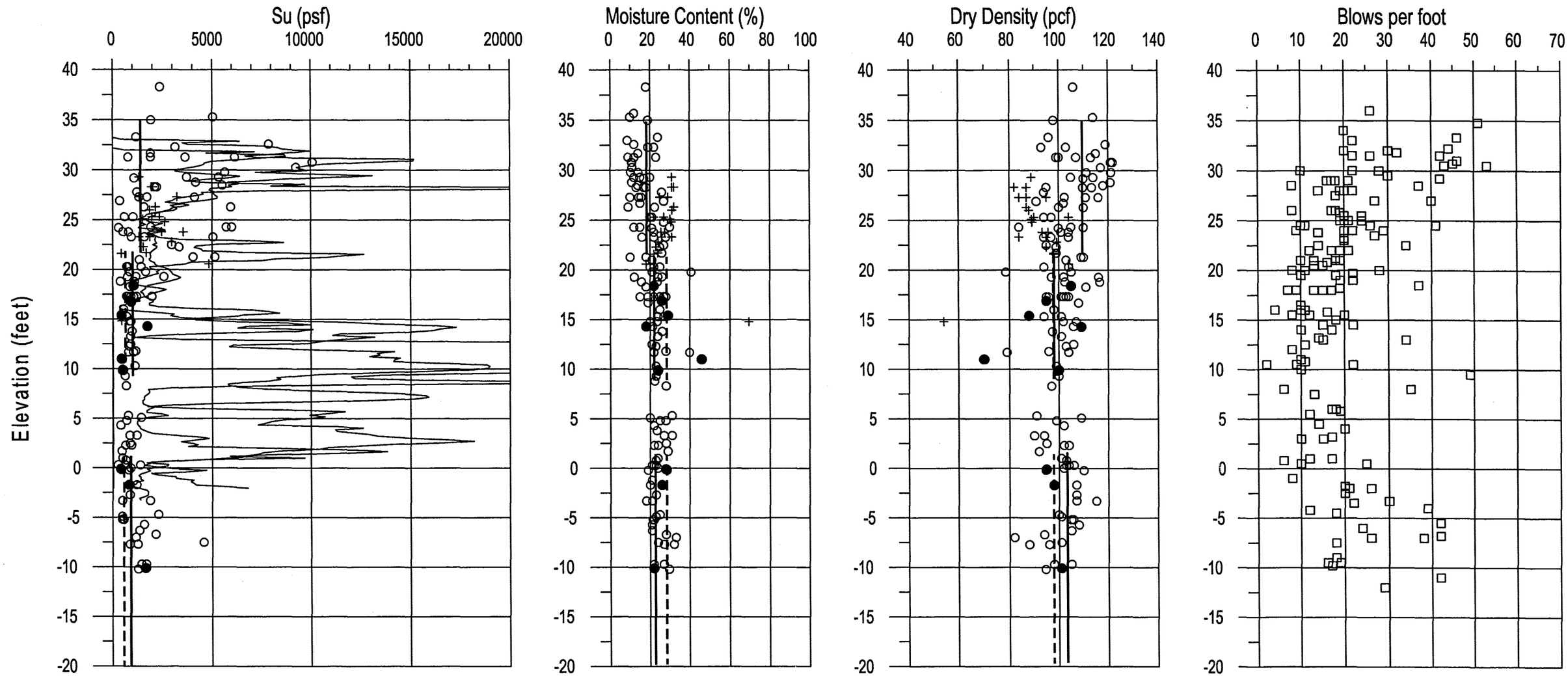
LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index27.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 27	Figure 35
7/15/02	URS		



Data from the following borings and CPTs:

AA-1	RBPS-58	SJIA-51	SJIA-S12	WB-14
AA-2	SJIA-24	SJIA-52	SJIA-S3	WB-23
AA-3	SJIA-26	SJIA-53	SJIA-S4	WB-24
AA-4	SJIA-31	SJIA-54	EB-15	CB-09
AA-5	SJIA-33	SJIA-55	EB-16	CB-10
RBPS-56	SJIA-34	SJIA-S10	EB-25	WC-7
RBPS-57	SJIA-50	SJIA-S11	EB-26	WC-8

Levee Fill and Foundation Soil

+ Fat Clay

○ Lean Clay

□ All Soil Types

Channel Sediments

+ Fat Clay

● Lean Clay

■ All Soil Types

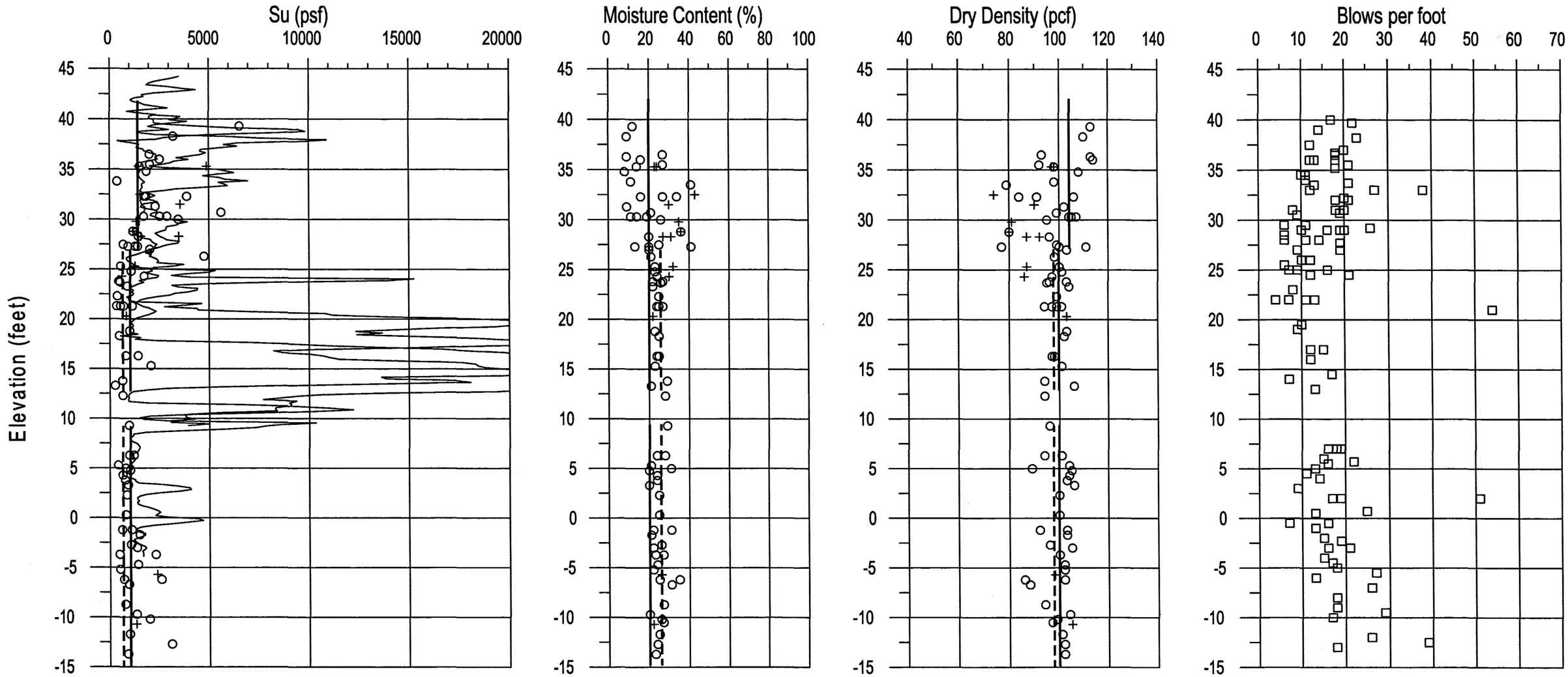
LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index28.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 28	Figure 36
7/15/02	URS		



Data from the following borings and CPTs:

SJIA-1	SJIA-3	SJIA-5	CB-09
SJIA-11	SJIA-44	SJIA-8	CB-10
SJIA-13	SJIA-45	SJIA-S1	EC-13
SJIA-16	SJIA-46	SJIA-S2	WC-09
SJIA-2	SJIA-48	EB-17	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

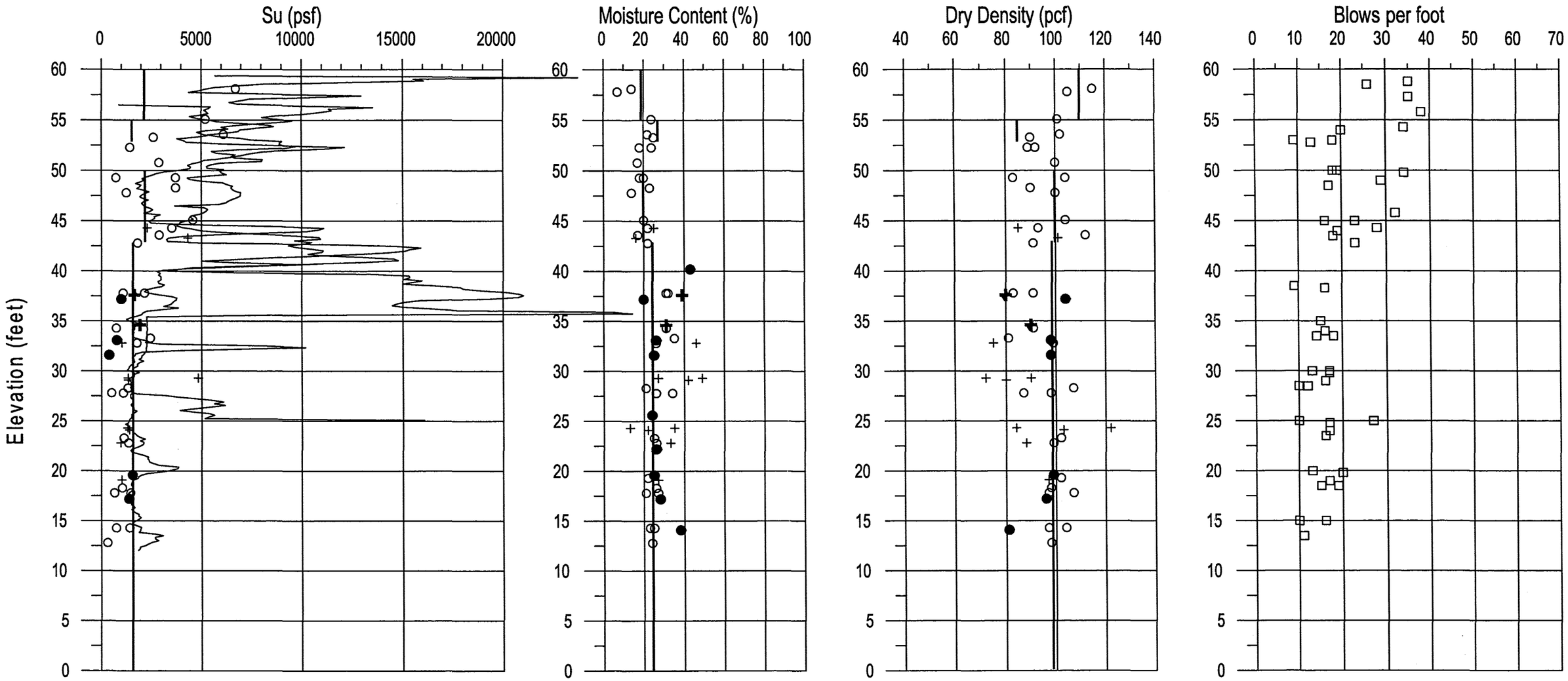
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index29.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 29	Figure 37
7/15/02			
	URS		



Data from the following borings and CPTs:

RHM-R4	WB-17
RHM-R5	CB-12
RHM-R6	CB-13
RHM-R7	WC-14
RHM-R8	WC-15

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\245\update\index30.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 30	Figure 38
7/12/02	URS		

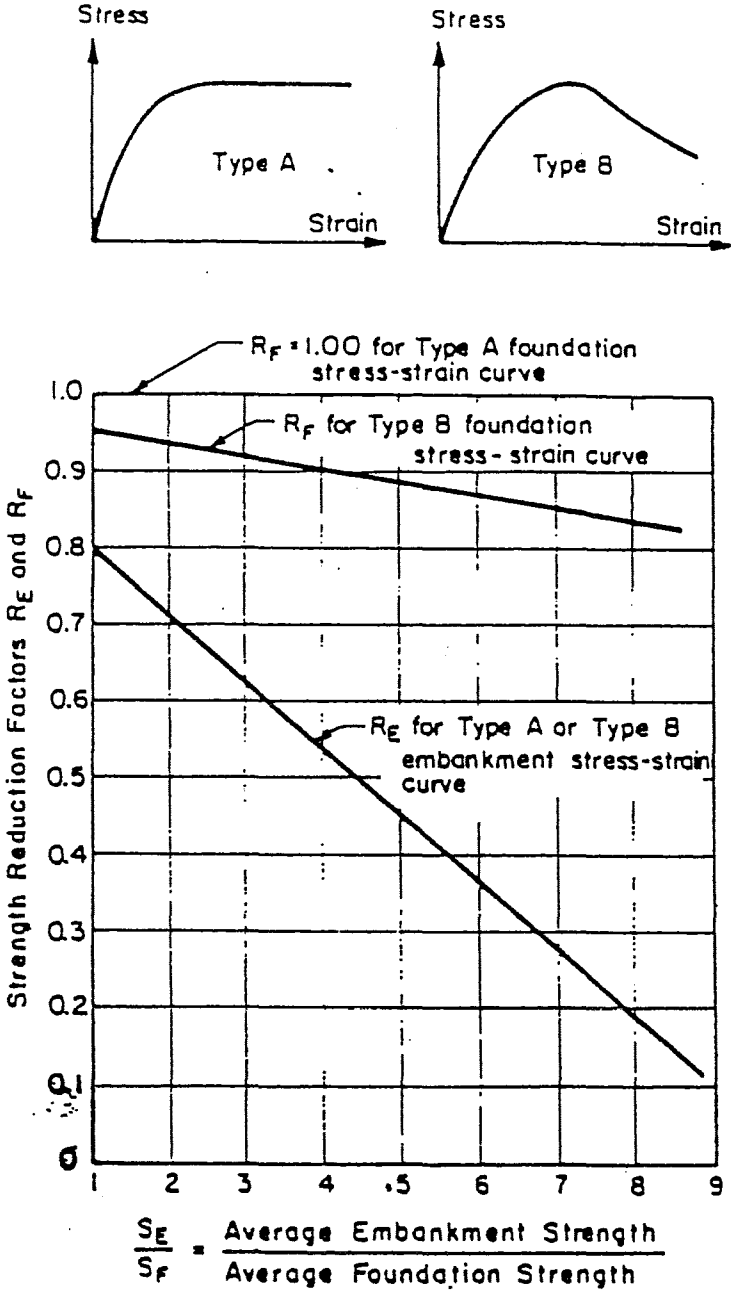
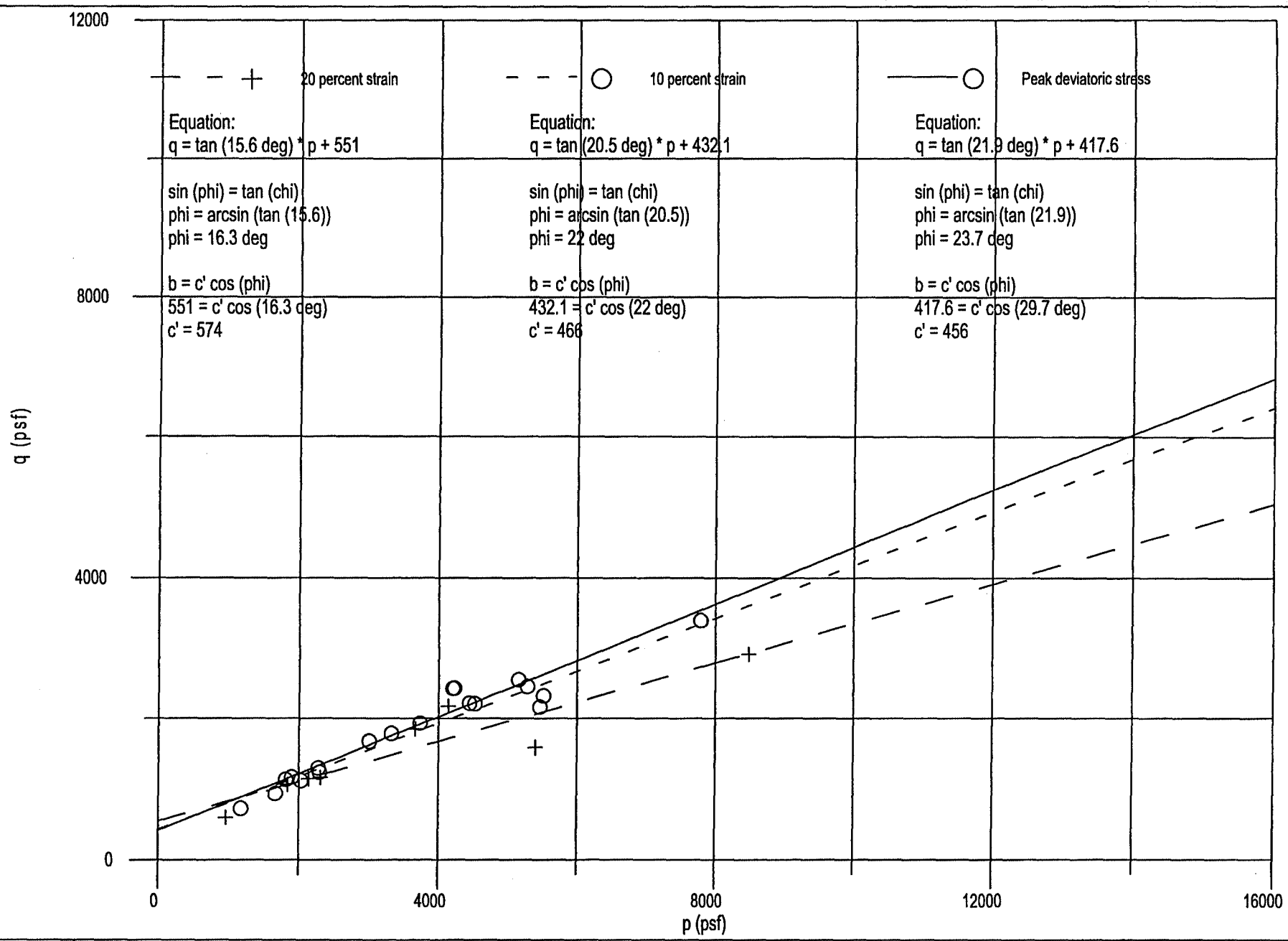


Fig. 3 CORRECTION FACTORS R_E AND R_F TO ACCOUNT FOR PROGRESSIVE FAILURE IN EMBANKMENTS ON SOFT CLAY FOUNDATIONS.
(after Chirapuntu and Duncan, 1975)

Q:\CADD2\01FILES\95-00112030.00\245\update\guadFig39.dwg

Project No. 5-00112030.00 7/15/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	EMBANKMENT STRENGTH REDUCTION FACTOR FOR SOFT CLAY FOUNDATIONS	Figure 39
URS			



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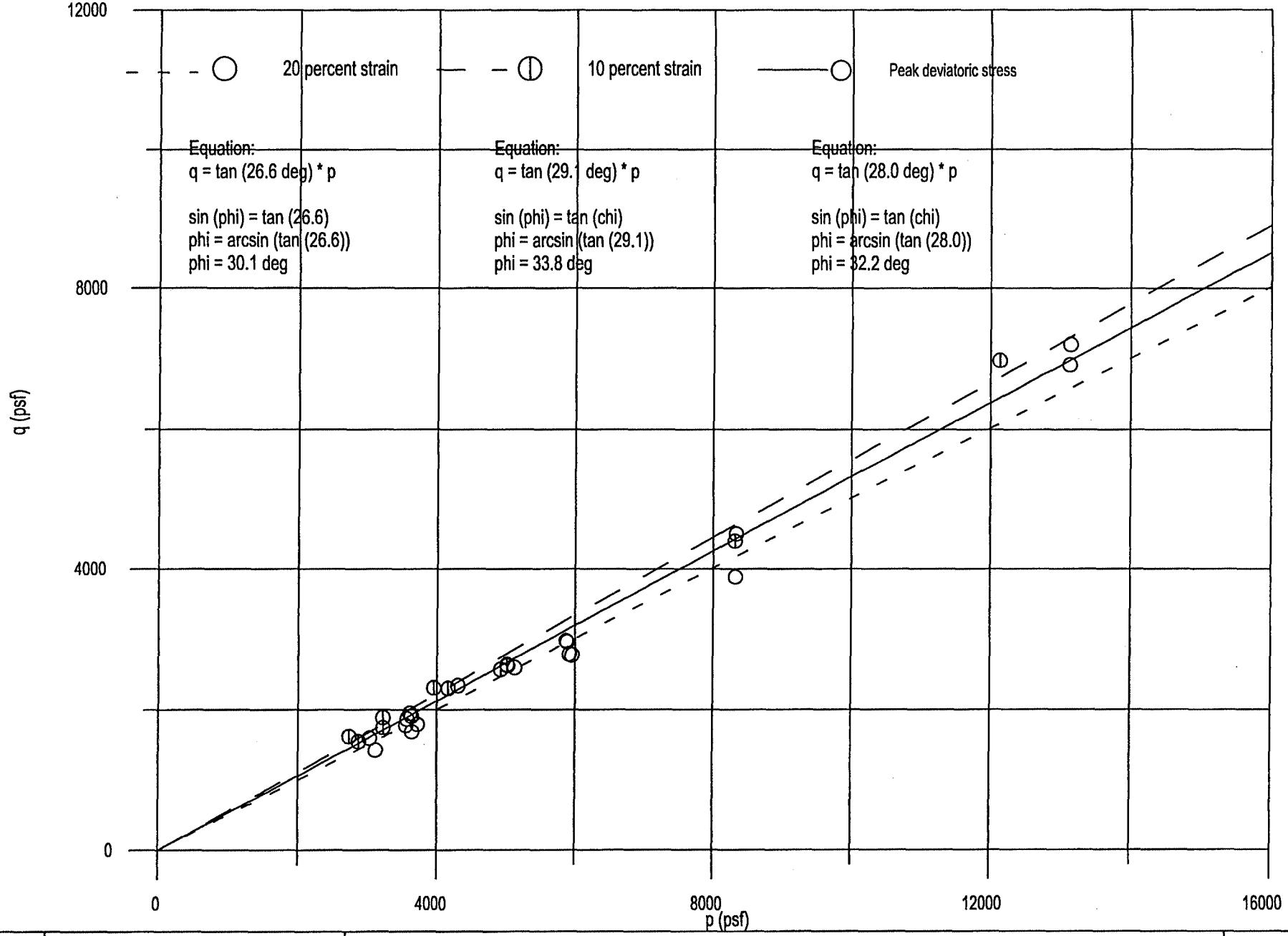
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	**Shear Strength Parameters For Lean Clay Fill with Dry Density less than 110 pcf**	Figure 40
7/15/02			
URS			

TEMP001



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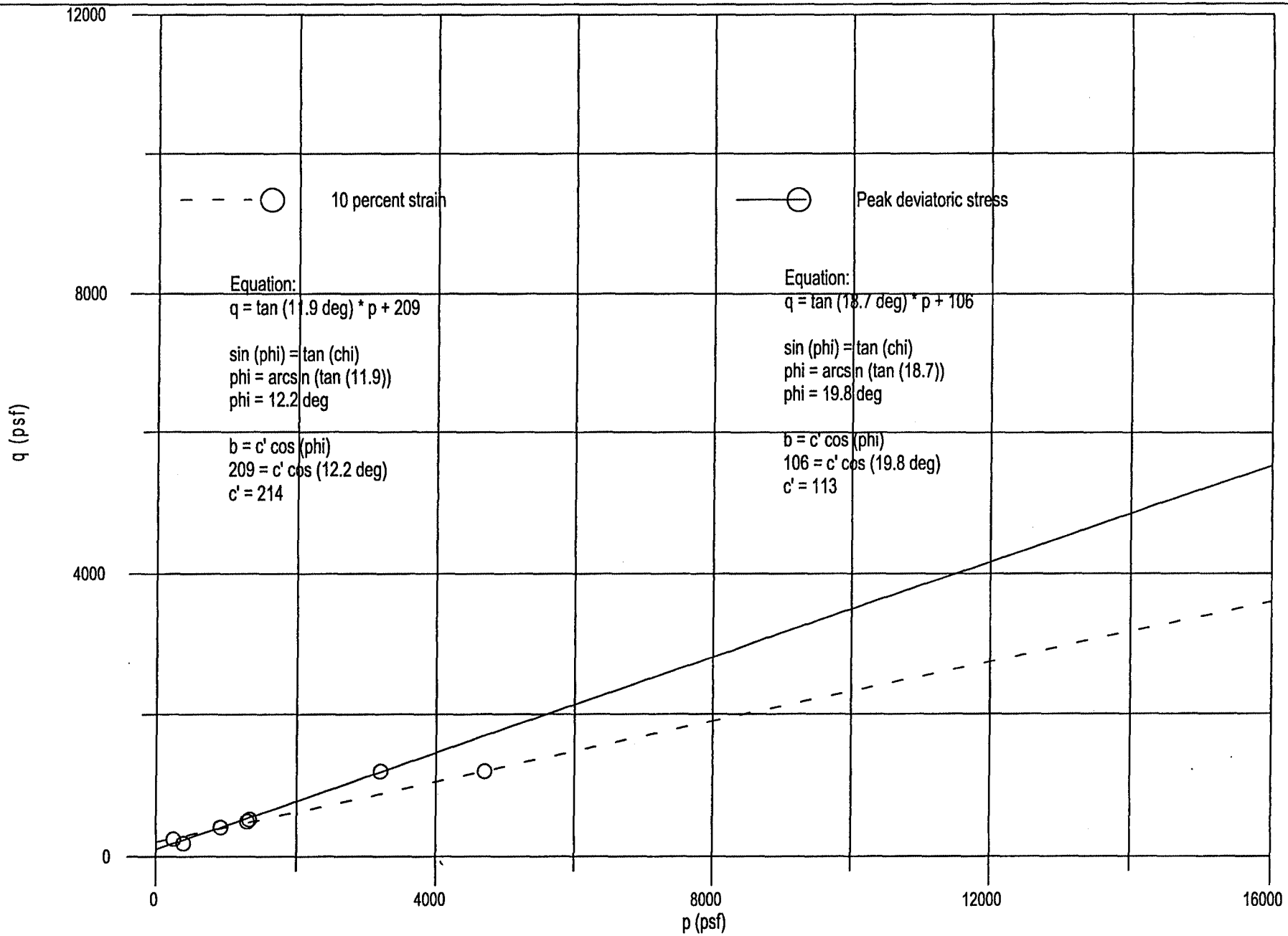
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	**Shear Strength Parameters For Lean Clay Fill with Dry Density greater than 110 pcf**	Figure 41
7/15/02			
URS			

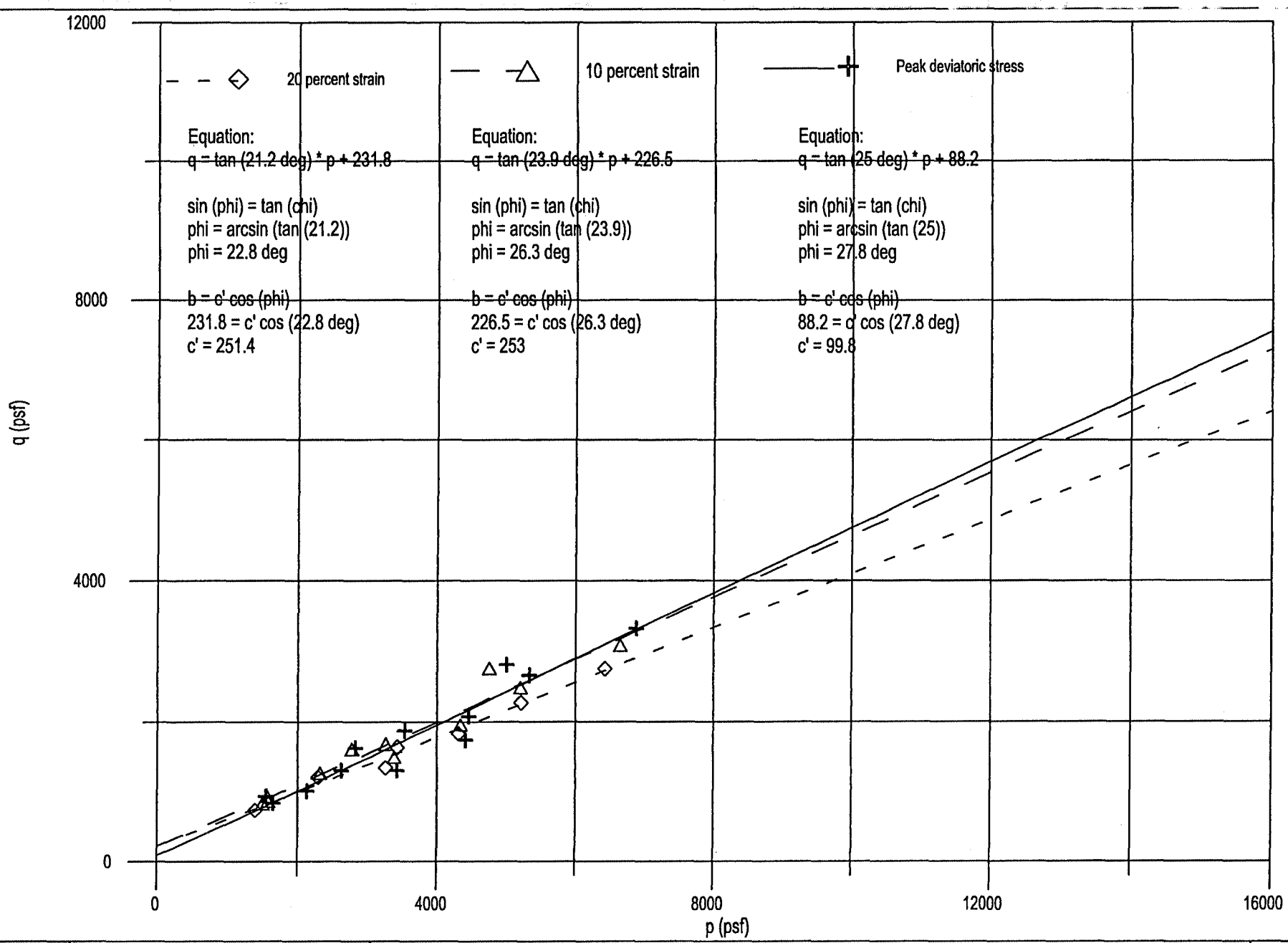
TEMP001



Q:\CADD2\01FILES\95-00112030.00\update\guadFig12.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	**Shear Strength Parameters For Fat Clay with Dry Density less than 80 pcf**	Figure 42
7/15/02			
URS			

TEMP001



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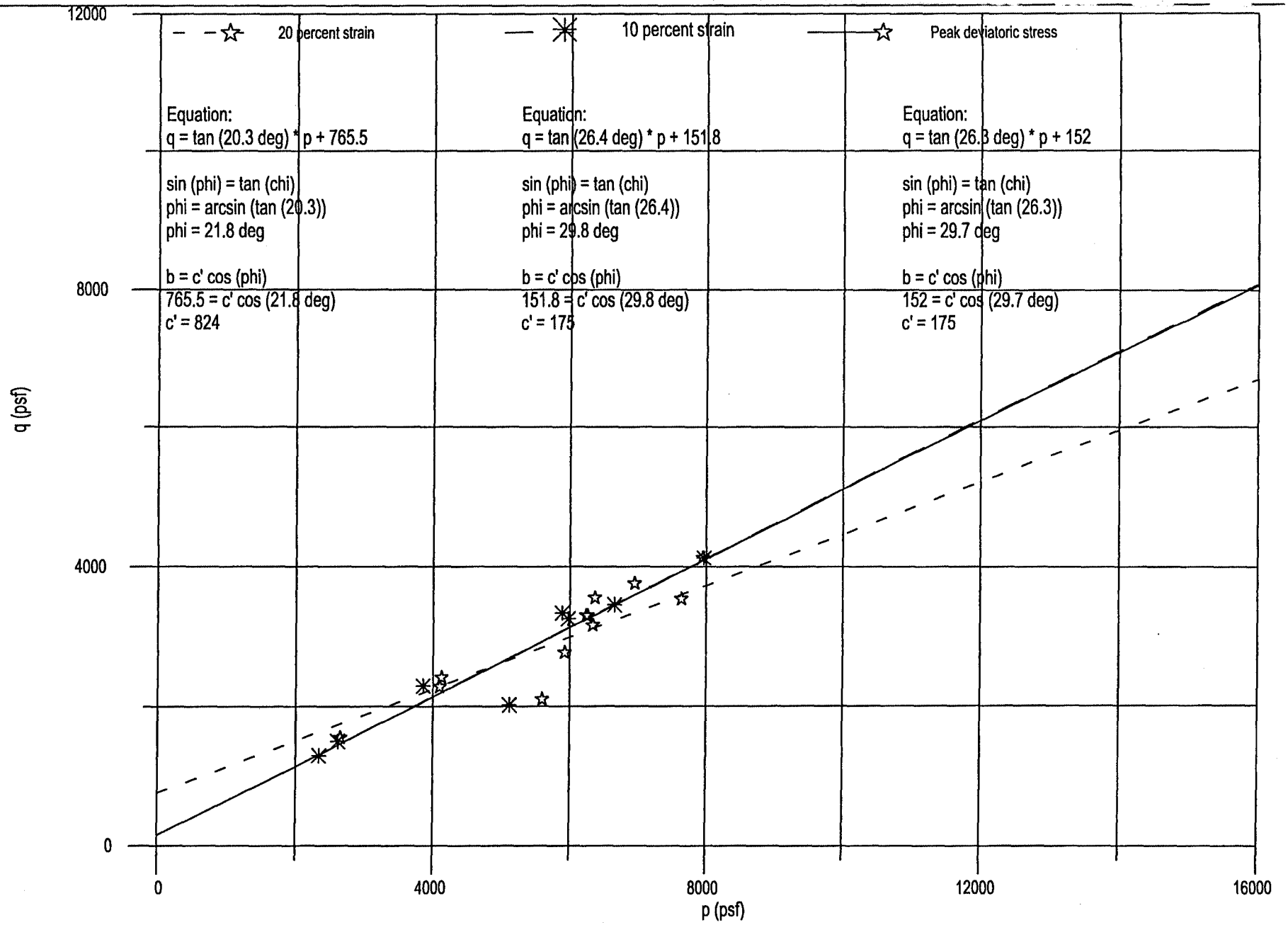
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Shear Strength Parameters For Lean Clay with Dry Density between 80 pcf and 100 pcf	Figure 43
7/15/02			
URS			

TEMP001



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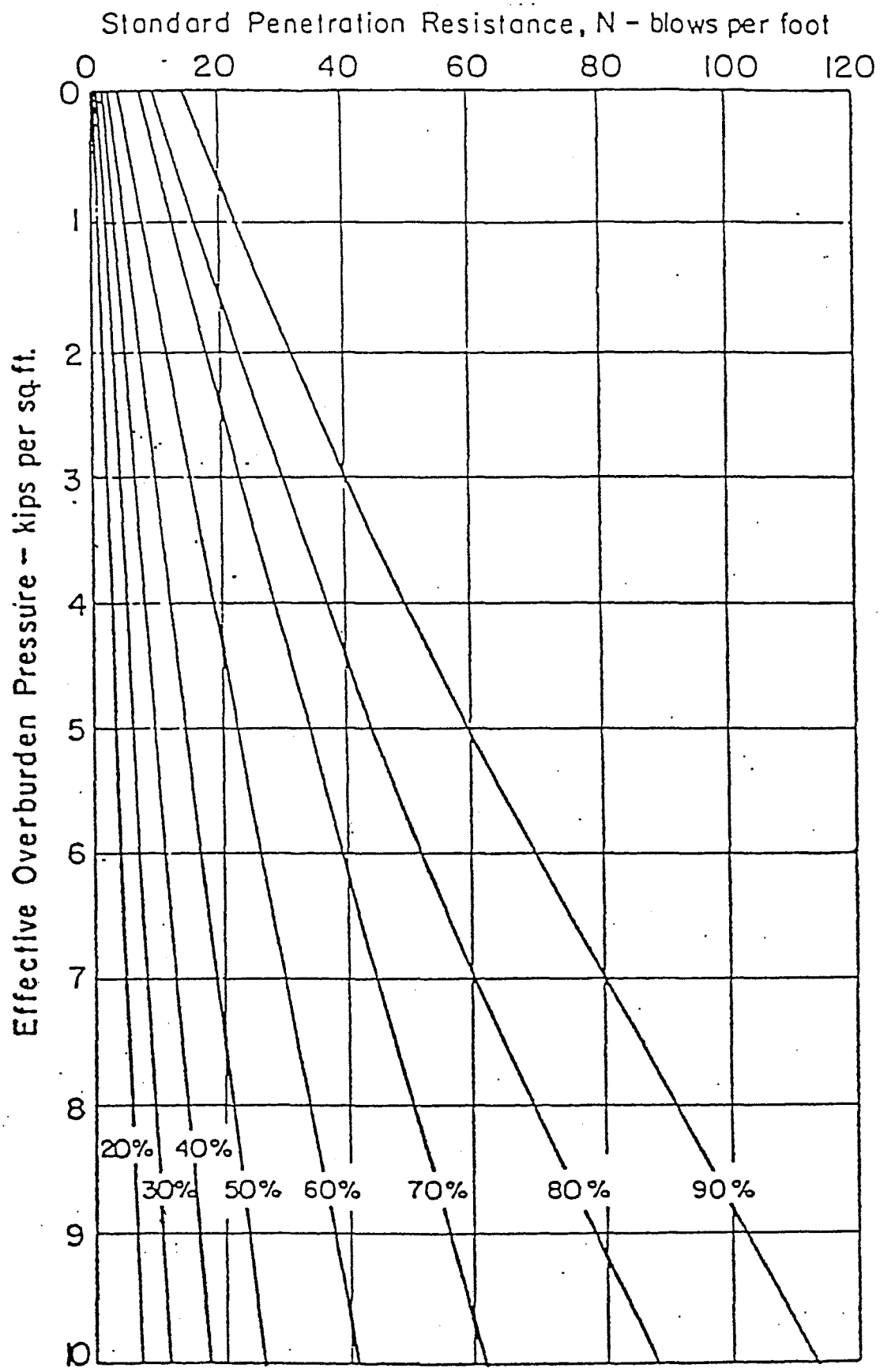
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	**Shear Strength Parameters For Lean Clay with Dry Density between 100 pcf and 110 pcf**	Figure 44
7/15/02			
URS			

TEMP001



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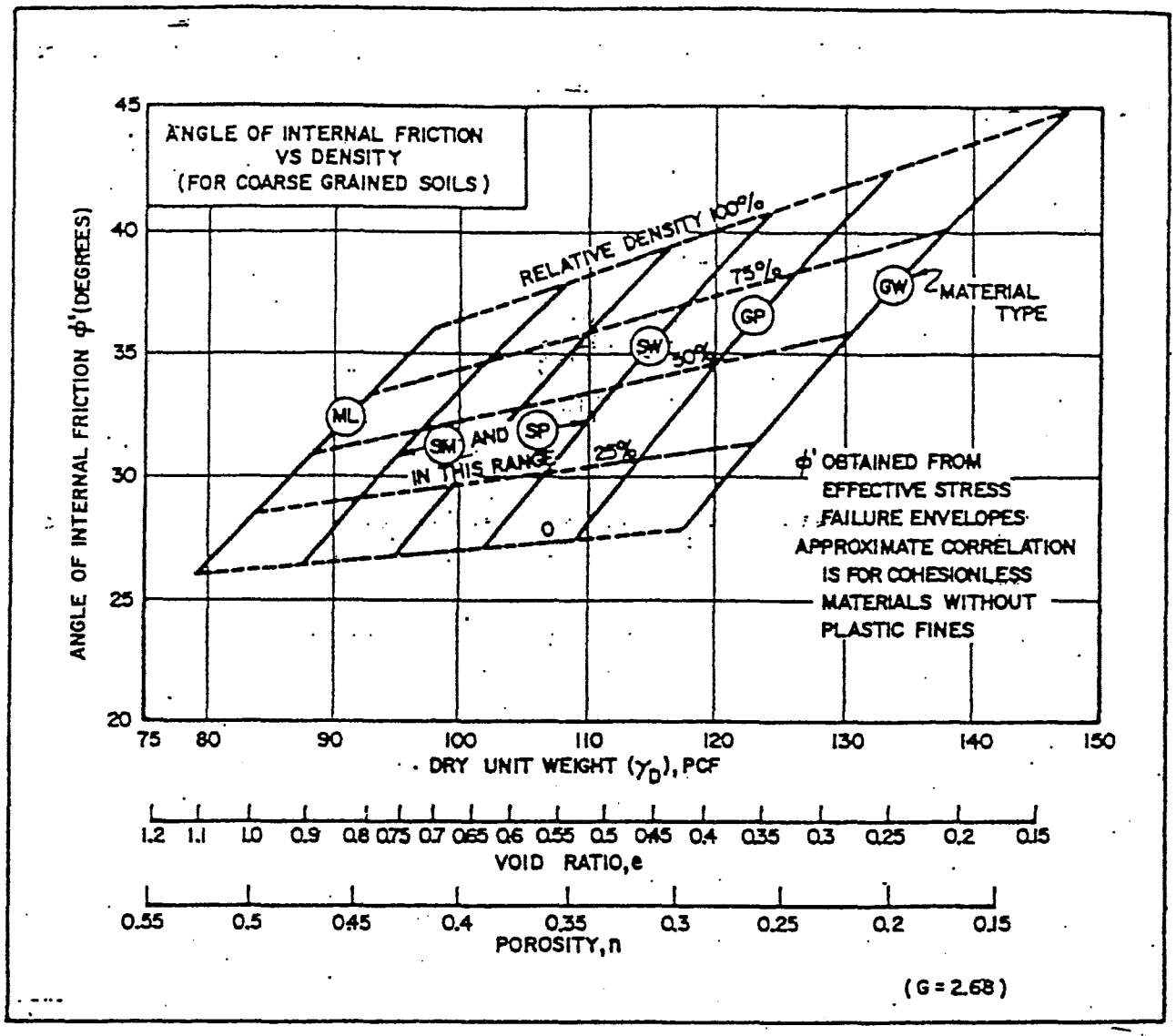
Project No. 5-00112030.00 7/15/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	**Relationship between Standard Penetration Resistance, Relative Density and Effective Overburden Pressure** (AFTER GIBBS AND HOLTZ, 1957)	Figure 45
URS			

TEMP002



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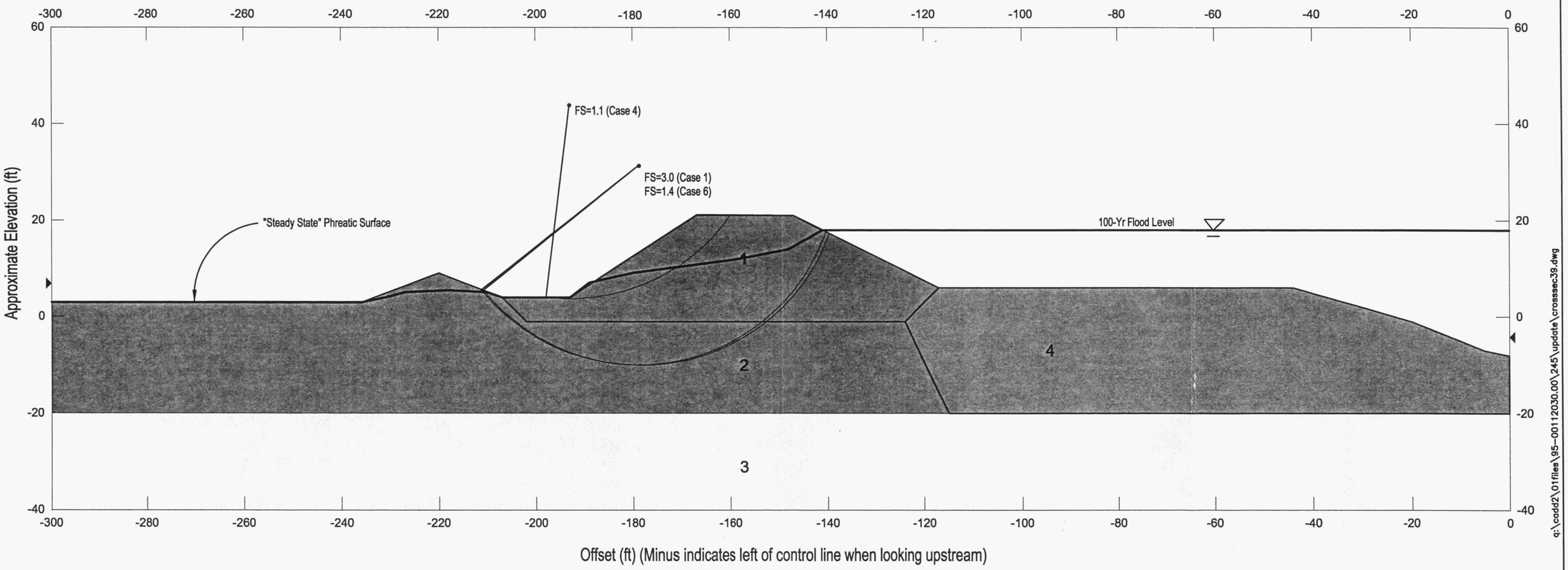
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	Correlation of Strength Characteristics for Granular Soils (NAVFAC, 1986)	Figure 46
7/15/02	Santa Clara County, California		

URS

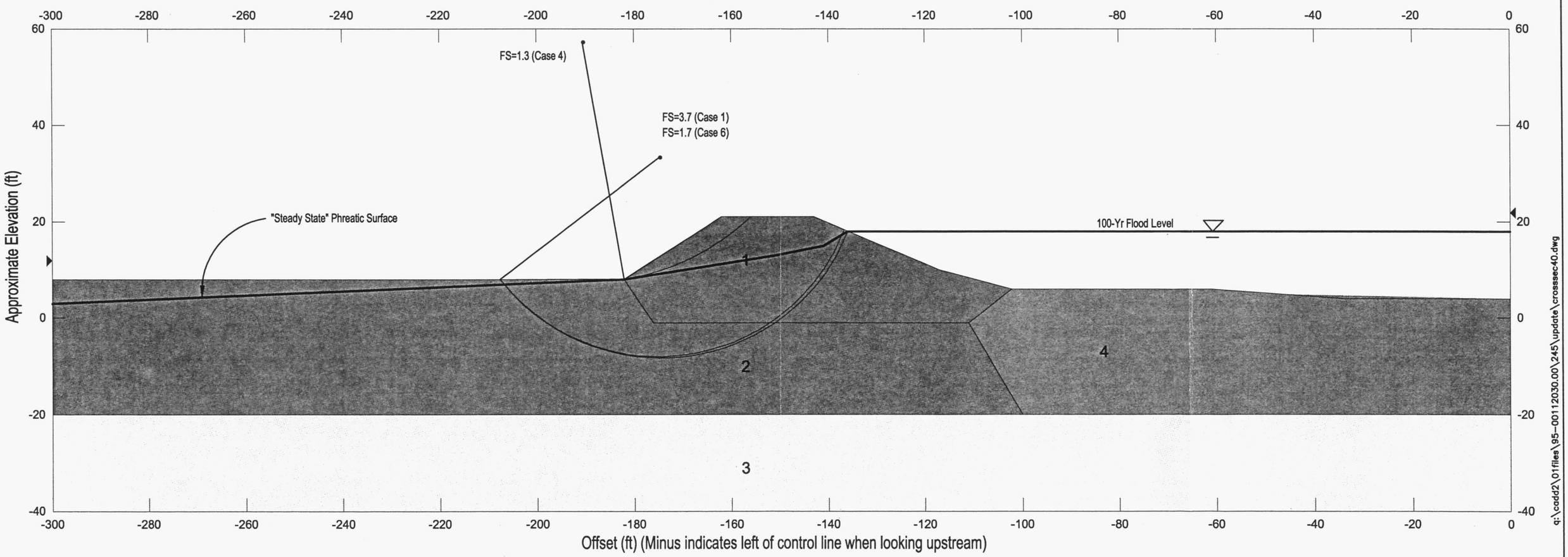
TEMP002

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		EB-09				23	750		
		Figure 23	110	21	133			2000	
1	Lean CLAY (FILL)		110	21	133	33	0	1420	Selected
		Table 6				30	35		
		EB-09				18	640		
		Figure 23	98	33	130			1000	
2	Lean CLAY		98	33	130	30	35	1000	Selected
						36	0		Estimated from N = 30, σ_v' = 3.0 ksf, Dr = 80%
3	Silty SAND				135	36	0		Selected
4	Lean CLAY (Channel Sediments)	CB-03, CB-04	94	26	118	30	35	775	Selected



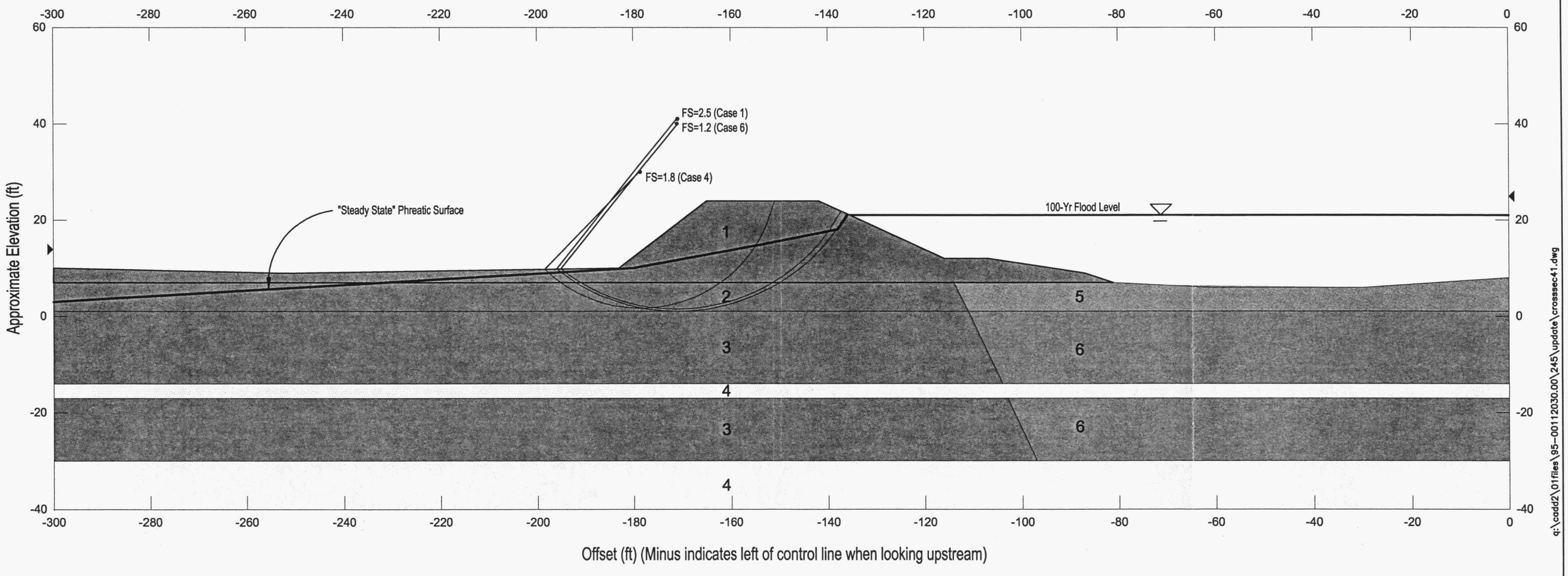
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 39 STATION 8+580 EAST LEVEE	Figure 47
TEMPCOM 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		EB-09				23	750		
		Figure 23	110	21	133			2000	
1	Lean CLAY (FILL)		110	21	133	33	0	1420	Selected
		Table 6				30	35		
		EB-09				18	640		
		Figure 23	98	33	130			1000	
2	Lean CLAY		98	33	130	30	35	1000	Selected
									Estimated from N = 30,
						36	0		σ_v' = 3.0 ksf, Dr = 80%
3	Silty SAND				135	36	0		Selected
4	Lean CLAY (Channel Sediments)	CB-03, CB-04	94	26	118	30	35	775	Selected



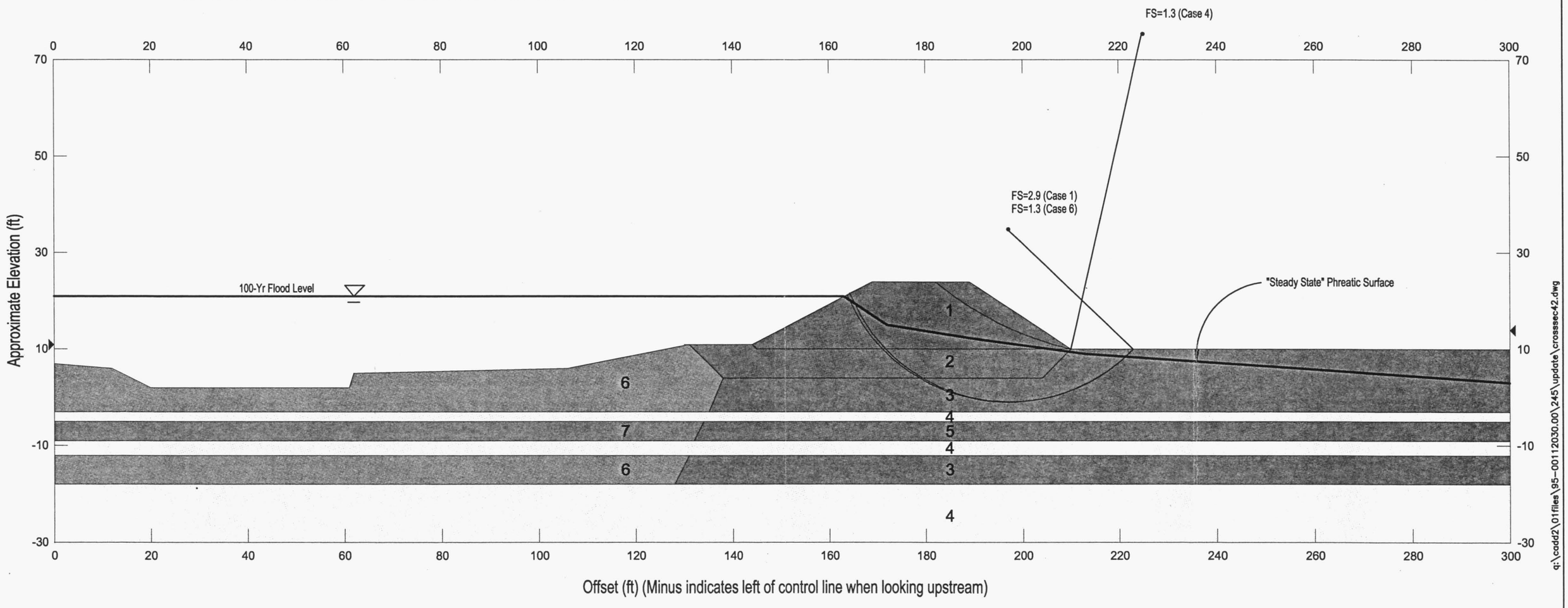
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 40 STATION 8+790 EAST LEVEE	Figure 48
TEMP004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		Figure 28	103	20	124			2500	
1	Lean CLAY (FILL)		103	20	124	24	450	1130	Selected
		Table 6				20	110		
		Figure 28	90	33	120			500	
2	Fat CLAY		90	33	120	20	110	500	Selected
		Table 6				30	150		
		GRIV-7				35	0		
		Figure 28	100	30	130			750	
3	Lean CLAY		100	30	130	30	150	750	Selected
						34	0		Estimated from N = 20, σ_v' = 1.9 ksf, Dr = 70%
4	Clayey SAND				135	34	0		Selected
5	Fat CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	20	110	500	Selected
6	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	150	750	Selected



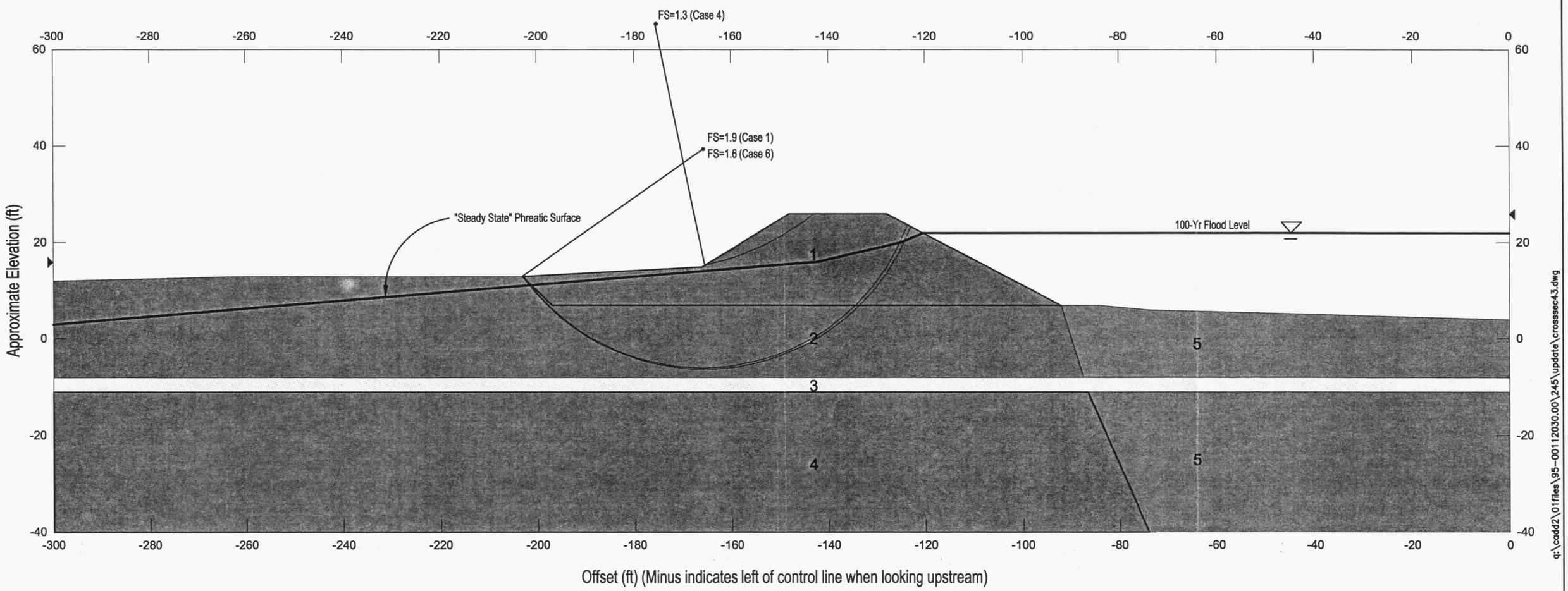
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 41 STATION 10+200 EAST LEVEE	Figure 49
TEMP004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 29	110 110	20 20	132 132	33 33	0 0	1800 1310	Selected
2	Lean CLAY (FILL)	Table 6 Figure 29	98 98	22 22	120 120	24 24	450 450	1000 800	Selected
3	Fat CLAY	Table 6 Figure 29	90 90	40 40	126 126	20 20	110 110	800 800	Selected
4	Clayey SAND				135	34 34	0 0		Estimated from N = 13, σ_v' = 1.6 ksf, Dr = 70% Selected
5	Lean CLAY	Table 6 Figure 29	100 100	30 30	130 130	30 30	150 150	1000 1000	Selected
6	Fat CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	20	110	800	Selected
7	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	150	900	Selected



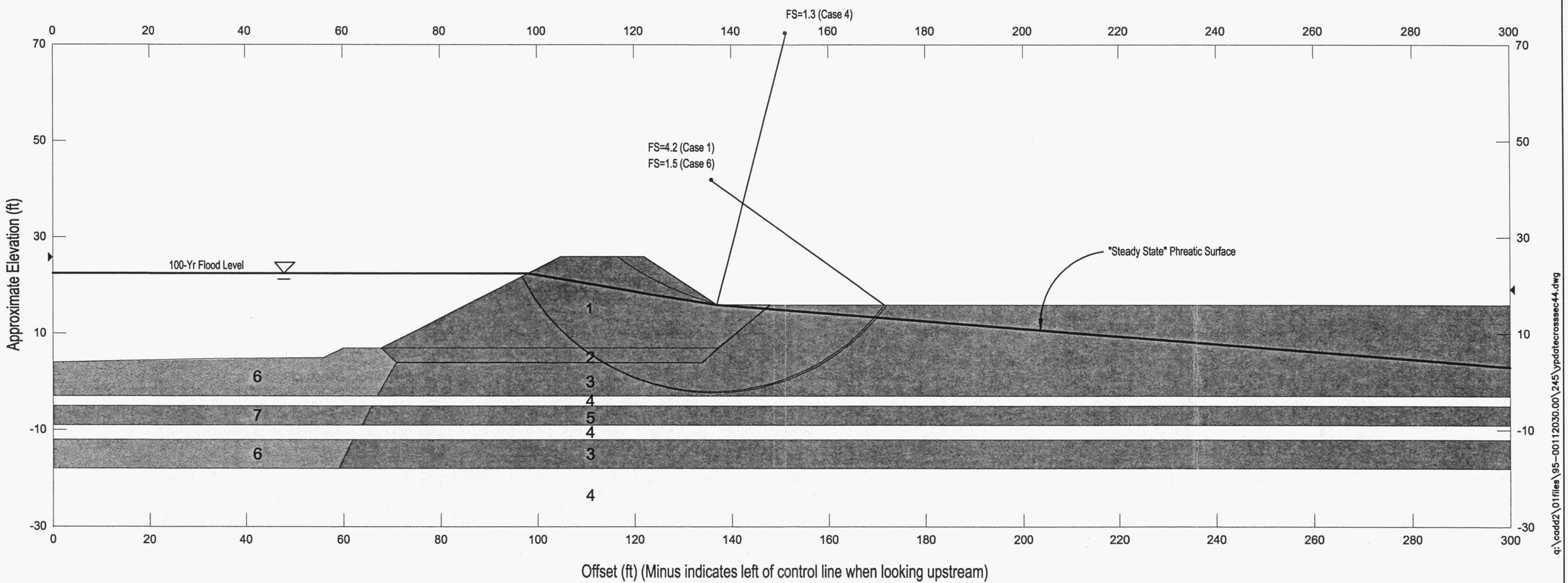
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 42 STATION 10+200 WEST LEVEE	Figure 50
TEMPPOOH 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		Figure 30	110	20	132			2000	
1	Lean CLAY (FILL)		110	20	132	33	0	1420	Selected
		Table 6				30	35		
		Figure 30	97	30	126			1000	
2	Lean CLAY		97	30	126	30	35	1000	Selected
						35	0		Estimated from N = 27,
3	Silty SAND				135	35	0		σ_v' = 2.0 ksf, Dr = 80% Selected
		Table 6				30	150		
		Figure 30	105	22	128			1000	
4	Lean CLAY		105	22	128	30	150	1000	Selected
5	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	150	1000	Selected



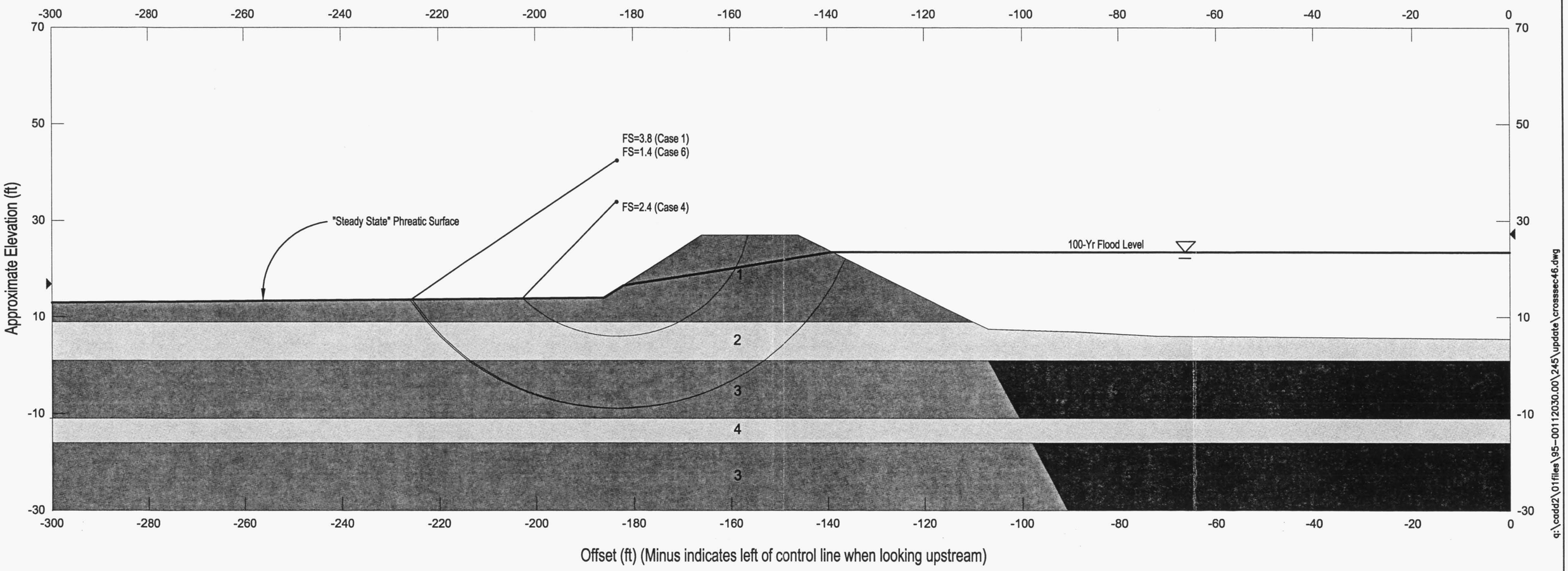
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 43 STATION 11+010 EAST LEVEE	Figure 51
TEMPO04 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 29	110 110	20 20	132 132	33 33	0 0	1800 1310	Selected
2	Lean CLAY (FILL)	Table 6 Figure 29	98 98	22 22	120 120	24 24	450 450	1000 800	Selected
3	Fat CLAY	Table 6 Figure 29	90 90	40 40	126 126	20 20	110 110	800 800	Selected
4	Clayey SAND				135	34 34	0 0		Estimated from N = 13, σ_v' = 1.6 ksf, Dr = 70% Selected
5	Lean CLAY	Table 6 Figure 29	100 100	30 30	130 130	30 30	150 150	1000 1000	Selected
6	Fat CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	20	110	800	Selected
7	Lean CLAY (Channel Sediments)	CB-05, CB-06	93	30	121	30	150	900	Selected



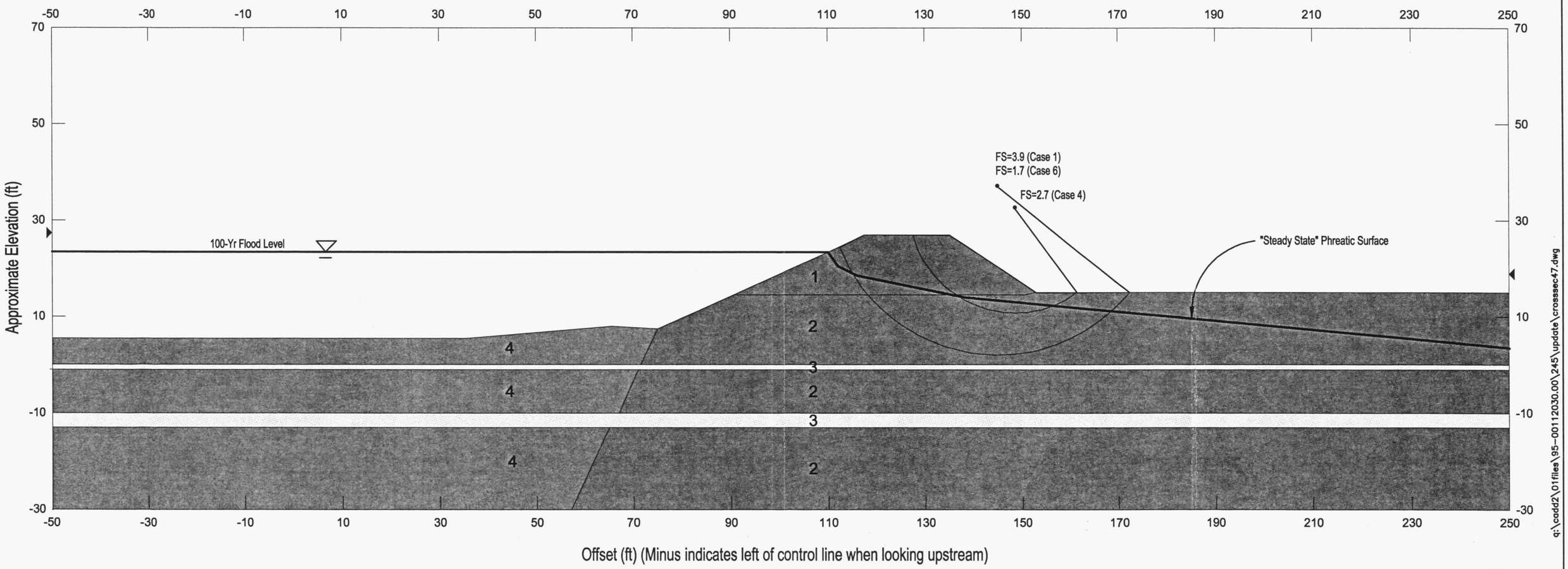
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 44 STATION 11+010 WEST LEVEE	Figure 52
TEMP004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		EB-11				31	140		
		Figure 31	98	20	118			2000	
1	Lean CLAY (FILL)		98	20	118	24	450	1420	Selected
						33	0		Estimated from N = 10, σ_v' = 1.6 ksf, Dr = 60%
2	Silty SAND				135	33	0		Selected
		Table 6				28	100		
		Figure 31	98	25	123			1000	
3	Lean CLAY		98	25	123	28	100	750	Selected
						33	0		Estimated from N = 7, σ_v' = 4.3 ksf, Dr = 40%
4	Silty SAND				135	33	0		Selected
5	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	35	500	Selected



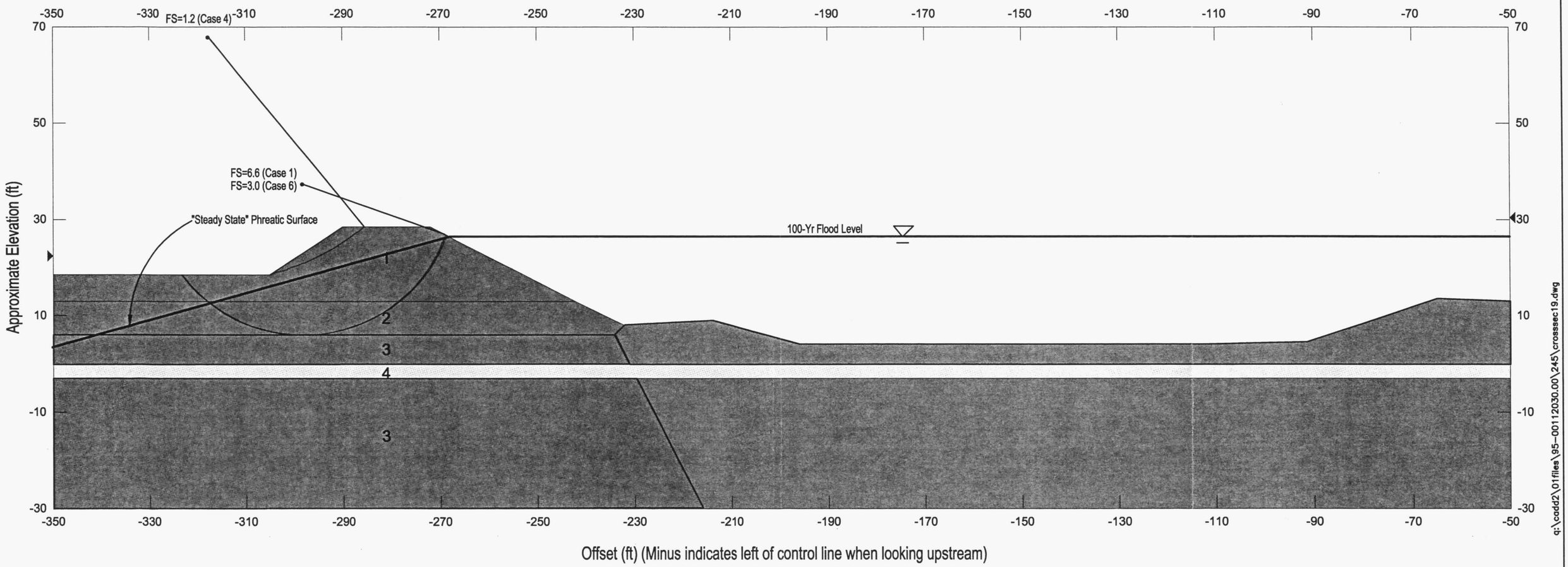
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 46 STATION 11+7610 EAST LEVEE	Figure 53
TEMP004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				24	450		
		WB-08				19	480		
		Figure 32	107	21	129			2000	
1	Lean CLAY (FILL)		107	21	129	24	450	1420	Selected
		Table 6				30	35		
		WB-08				25	170		
		Figure 32	95	32	125			1000	
2	Lean CLAY		95	32	125	30	35	1000	Selected
									Estimated from N = 19,
						35	0		σ_v' = 1.5 ksf, Dr = 80%
3	Silty SAND				135	35	0		Selected
4	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	35	500	Selected



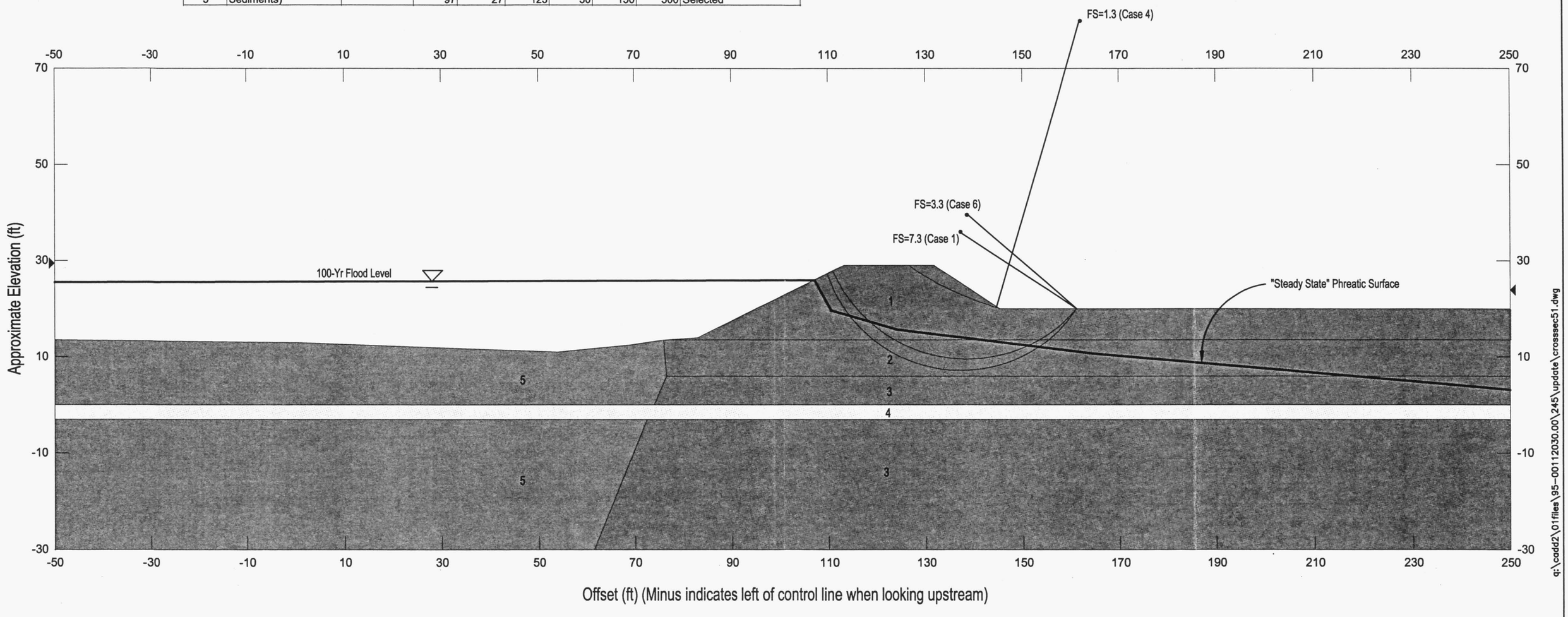
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5 H:1V OUTBOARD SLOPE CROSS SECTION 47 STATION 11+610 WEST LEVEE	Figure 54
TEMPO04 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		Figure 33	110	19	131			2000	
1	Lean CLAY (FILL)		110	19	131	33	0	1540	Selected
		Table 6				20	110		
		Figure 33	100	28	128			1500	
2	Fat CLAY		100	28	128	20	110	1500	Selected
		Table 6				30	150		
		Figure 33	107	20	128			1500	
3	Lean CLAY		107	20	128	30	150	1500	Selected
						34	0		Estimated from N = 20, σ_v' = 1.9 ksf, Dr = 70%
4	Silty SAND				135	34	0		Selected
5	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected



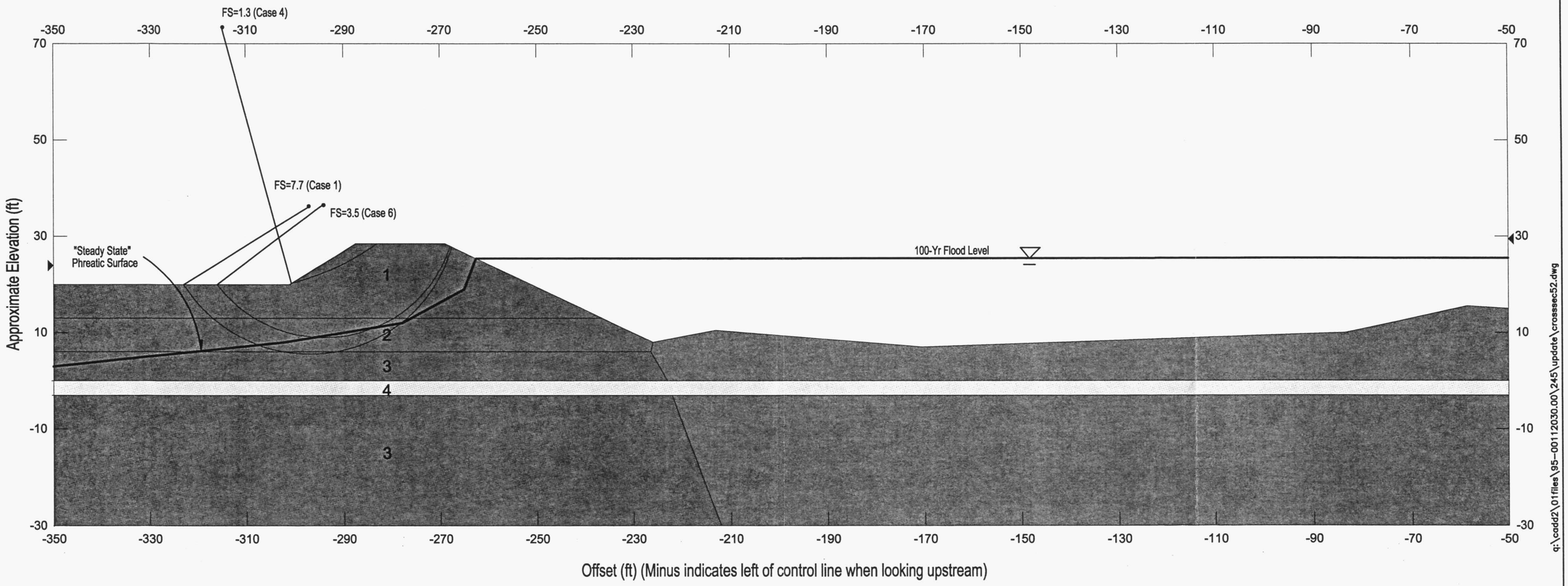
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 50 STATION 12+120 EAST LEVEE	Figure 55
TEMP004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		Figure 33	110	19	131			2000	
1	Lean CLAY (FILL)		110	19	131	33	0	1540	Selected
		Table 6				20	110		
		Figure 33	100	28	128			1500	
2	Fat CLAY		100	28	128	20	110	1500	Selected
		Table 6				30	150		
		Figure 33	107	20	128			1500	
3	Lean CLAY		107	20	128	30	150	1500	Selected
						34	0		Estimated from N = 20,
4	Silty SAND				135	34	0		σ_v' = 1.9 ksf, Dr = 70% Selected
5	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected



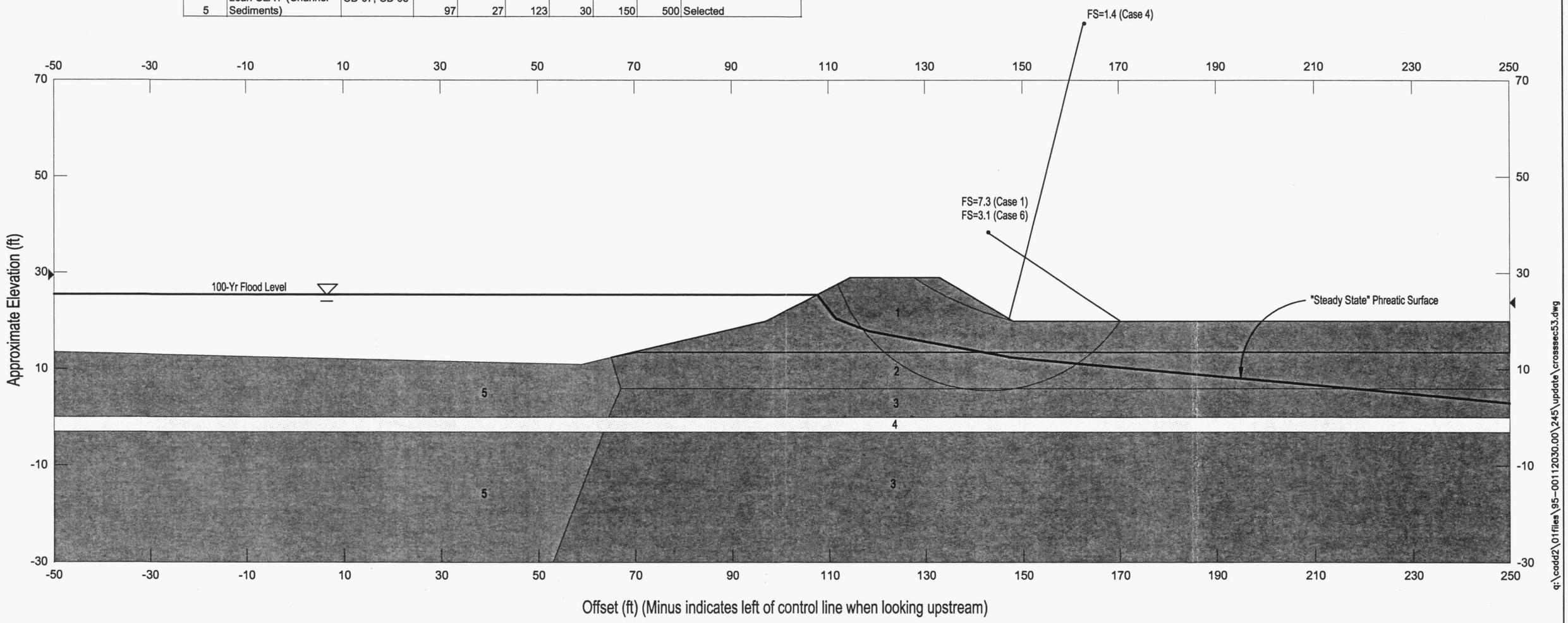
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 51 STATION 12+120 WEST LEVEE	Figure 56
TEMPF004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 33	110 110	19 19	131 131	33 33	0 0	2000 1540	Selected
2	Fat CLAY	Table 6 Figure 33	100 100	28 28	128 128	20 20	110 110	1500 1500	Selected
3	Lean CLAY	Table 6 Figure 33	107 107	20 20	128 128	30 30	150 150	1500 1500	Selected
4	Silty SAND				135	34 34	0 0		Estimated from N = 20, σ_v' = 1.9 ksf, Dr = 70% Selected
5	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected



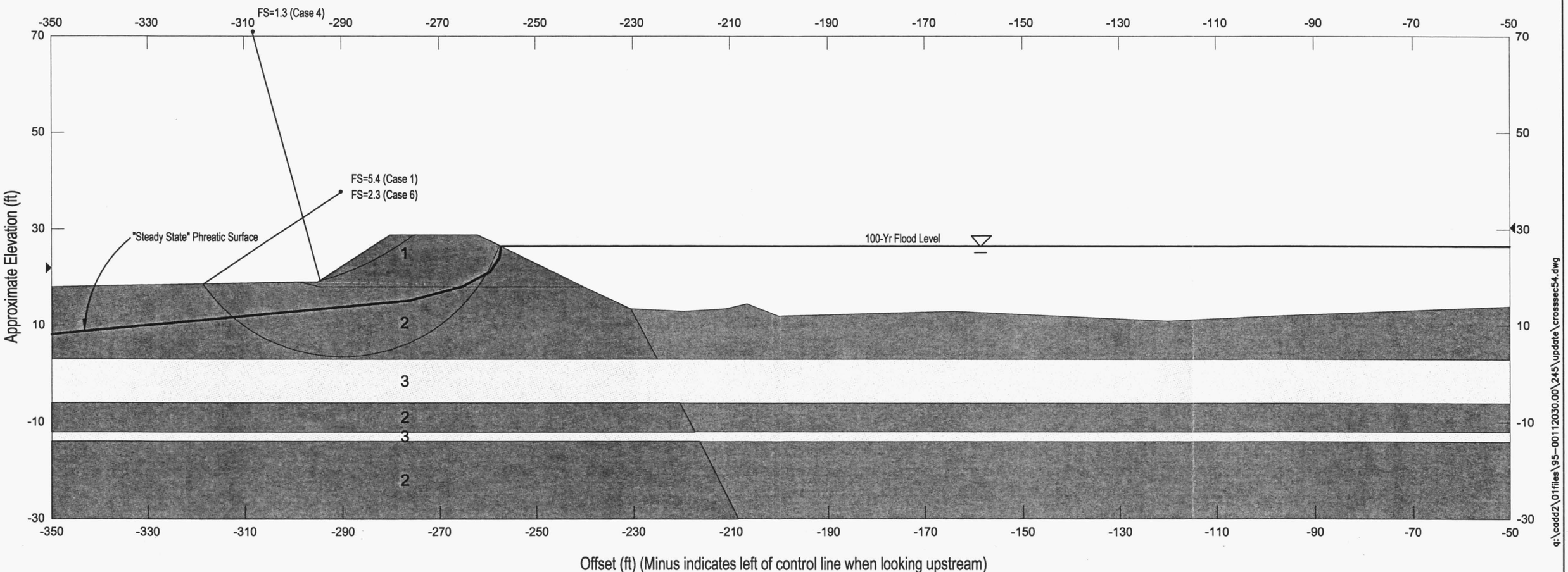
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUT BOARD SLOPE CROSS SECTION 52 STATION 12+300 EAST LEVEE	Figure 57
TEMP004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		Figure 33	110	19	131			2000	
1	Lean CLAY (FILL)		110	19	131	33	0	1540	Selected
		Table 6				20	110		
		Figure 33	100	28	128			1500	
2	Fat CLAY		100	28	128	20	110	1500	Selected
		Table 6				30	150		
		Figure 33	107	20	128			1500	
3	Lean CLAY		107	20	128	30	150	1500	Selected
						34	0		Estimated from N = 20, σ_v' = 1.9 ksf, Dr = 70%
4	Silty SAND				135	34	0		Selected
5	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected



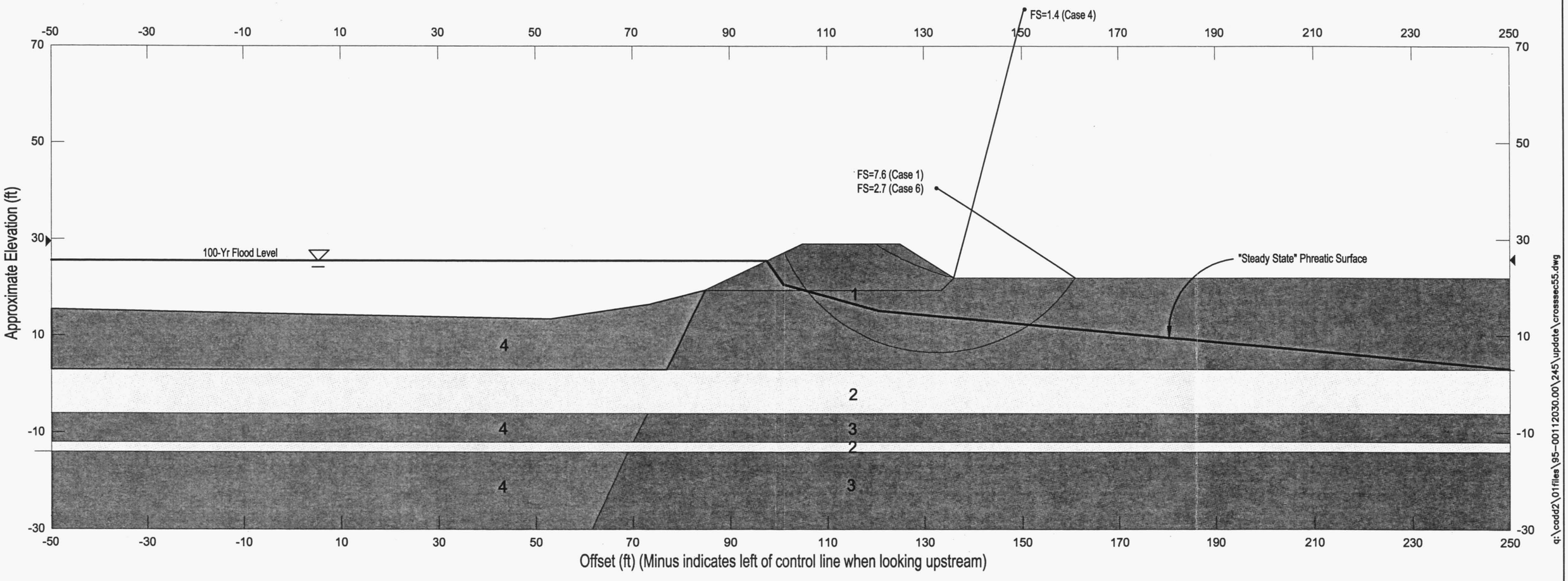
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUT BOARD SLOPE CROSS SECTION 53 STATION 12++300 WEST LEVEE	Figure 58
TEMP004 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		EB-14				36	320		
		Figure 34	110	18	130			2000	
1	Lean CLAY (FILL)		110	18	130	33	0	1460	Selected
		Table 6				30	150		
		Figure 34	100	25	125			1200	
2	Lean CLAY		100	25	125	30	150	1200	Selected
						35	0		Estimated from N = 35, σ_v' = 2.2 ksf, Dr = 90%
3	Silty SAND				135	35	0		Selected
4	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected

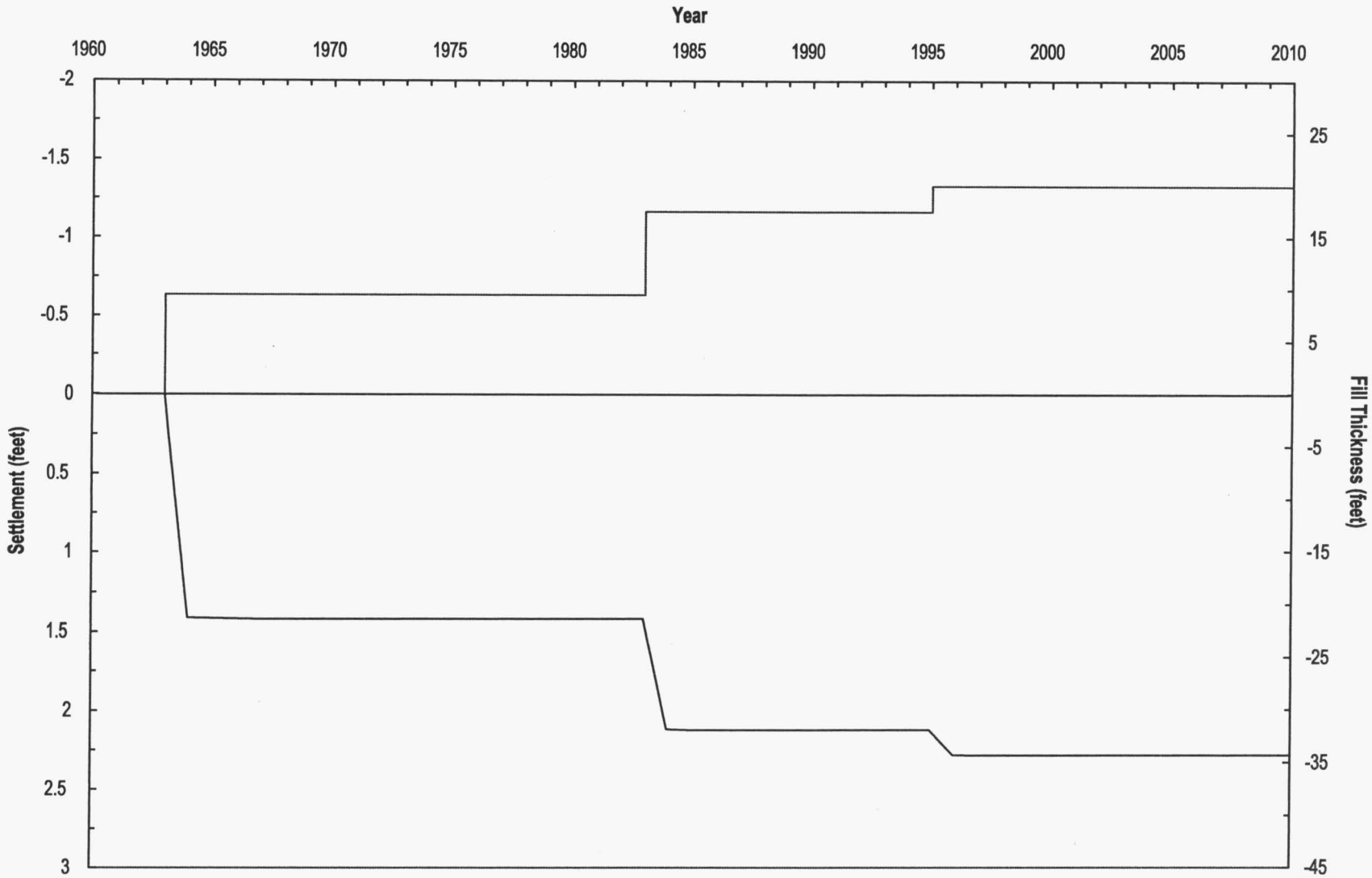


Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUT BOARD SLOPE CROSS SECTION 54 STATION 12+900 EAST LEVEE	Figure 59
TEMPO04 6/17/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 6				33	0		
		EB-14				36	320		
		Figure 34	110	18	130			2000	
1	Lean CLAY (FILL)		110	18	130	33	0	1460	Selected
		Table 6				30	150		
		Figure 34	100	25	125			1200	
2	Lean CLAY		100	25	125	30	150	1200	Selected
						35	0		Estimated from N = 35, σ_v' = 2.2 ksf, Dr = 90%
3	Silty SAND				135	35	0		Selected
4	Lean CLAY (Channel Sediments)	CB-07, CB-08	97	27	123	30	150	500	Selected



Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS 1.5H:1V OUTBOARD SLOPE CROSS SECTION 55 STATION 12+900 WEST LEVEE	Figure 60
TEMP004 6/17/02	URS		



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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 14	Figure 61
7/15/02			
URS			

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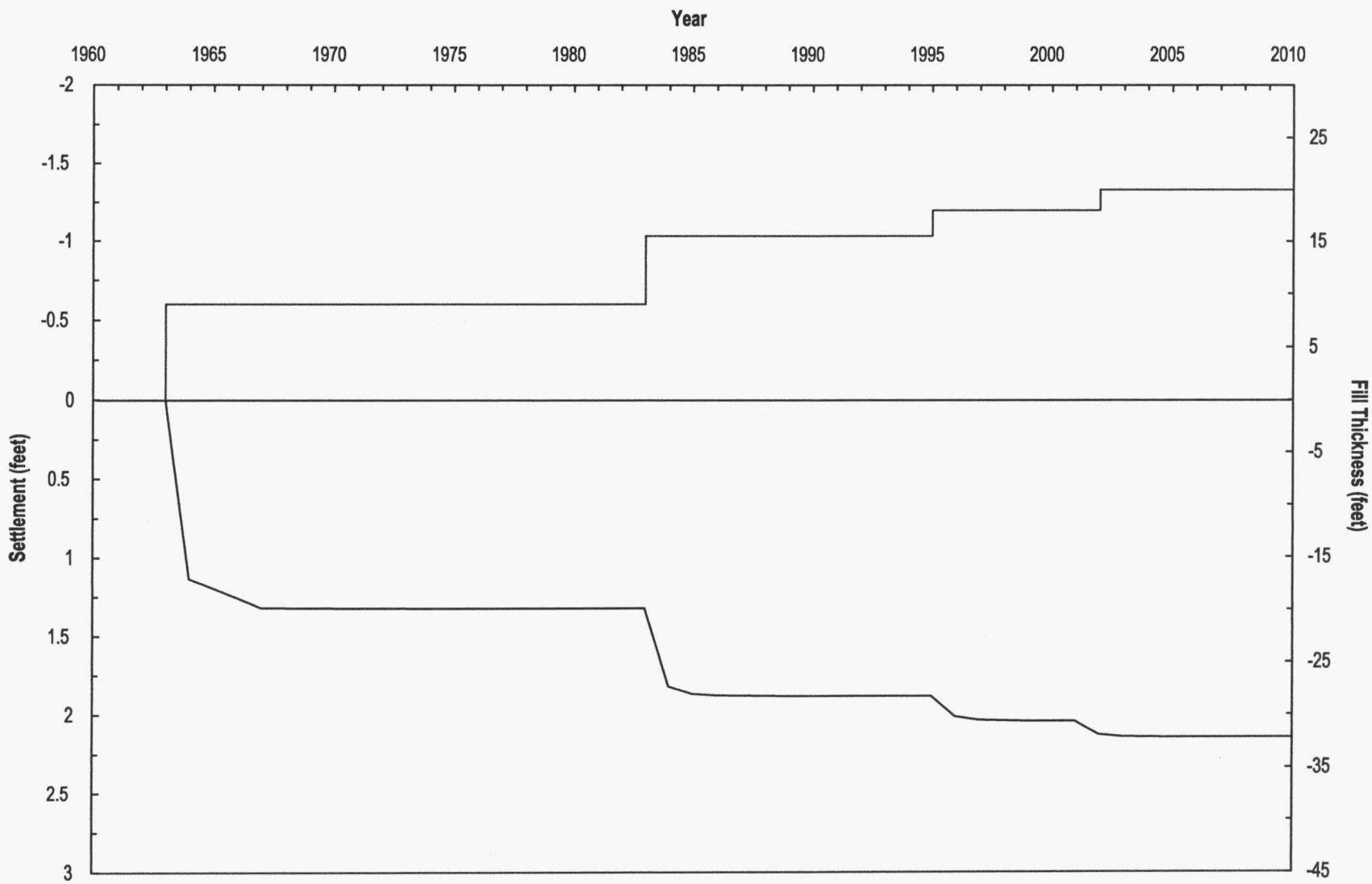

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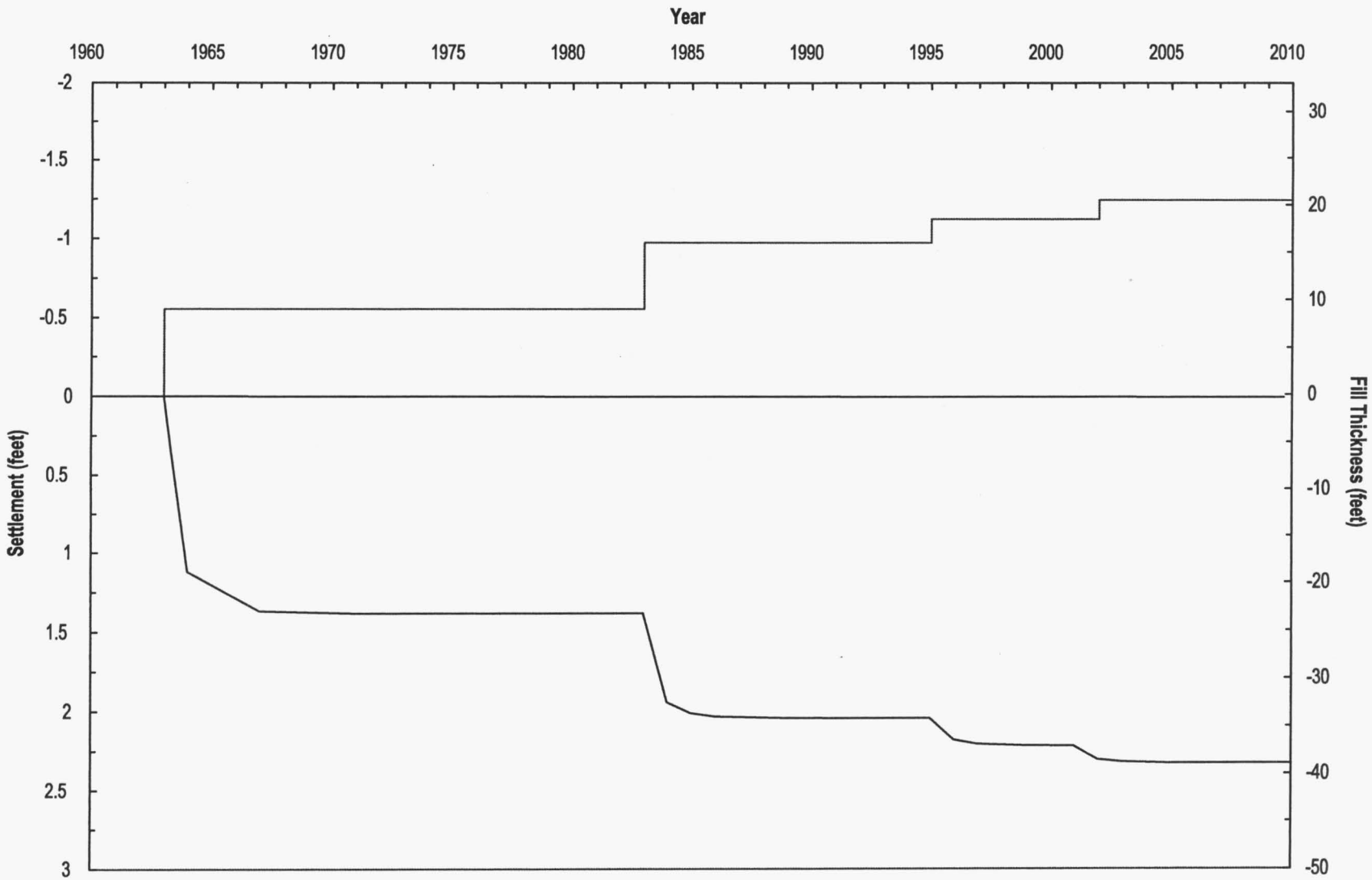
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 15
7/15/02		Figure 62
URS		

TEMP002



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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 16	Figure 63
7/15/02			
URS			

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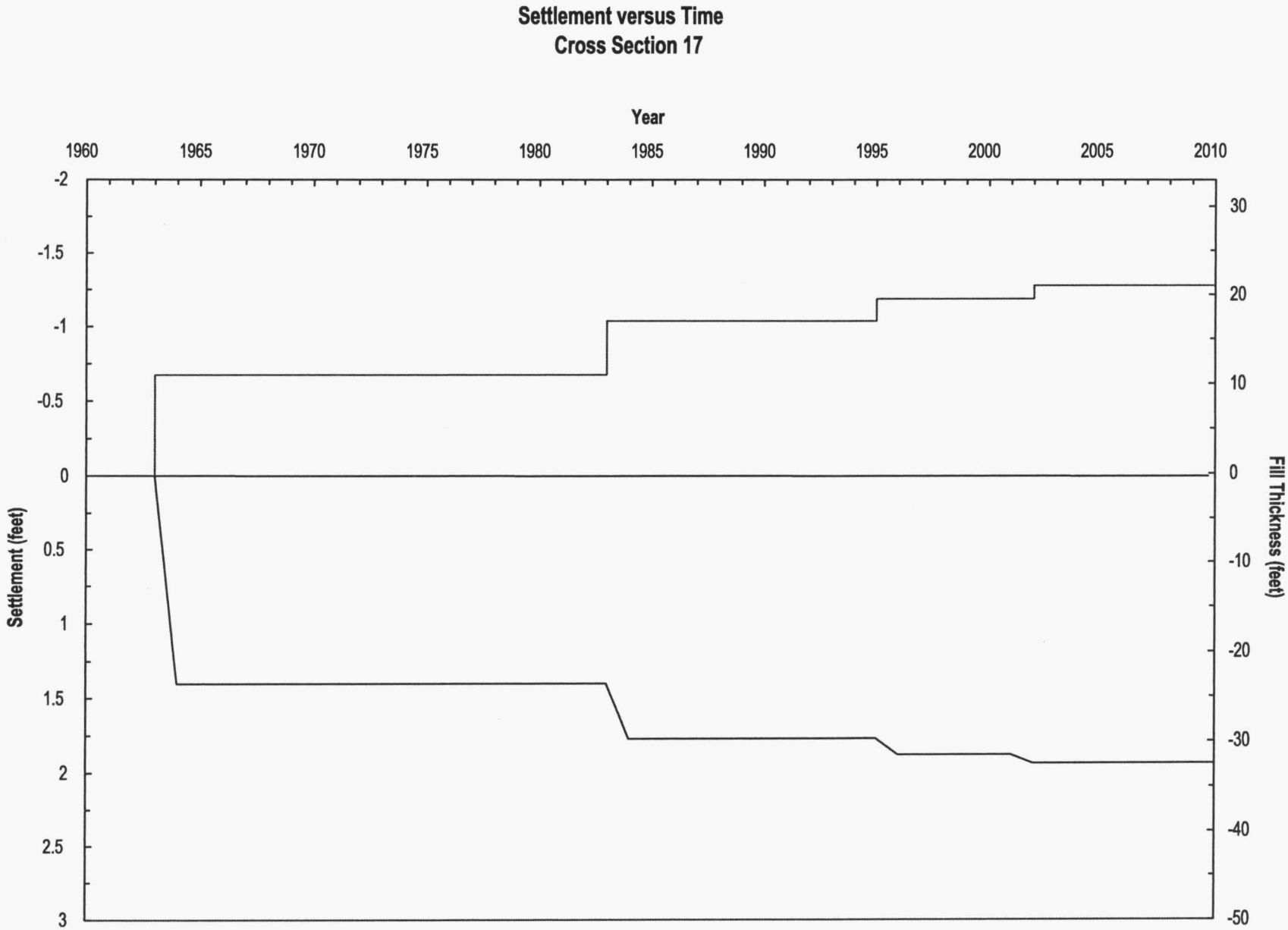

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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 17	Figure 64
7/15/02			
URS			

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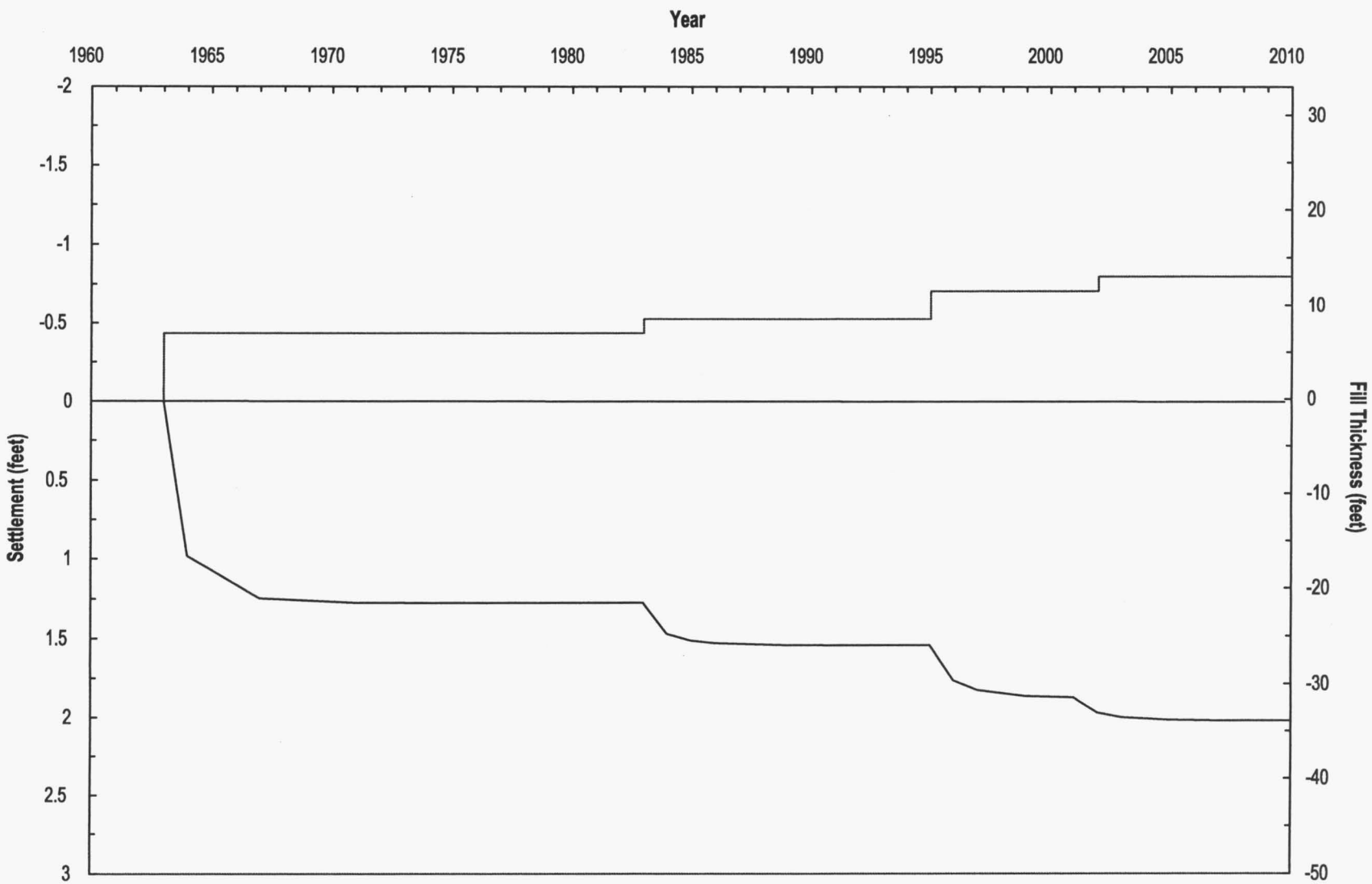

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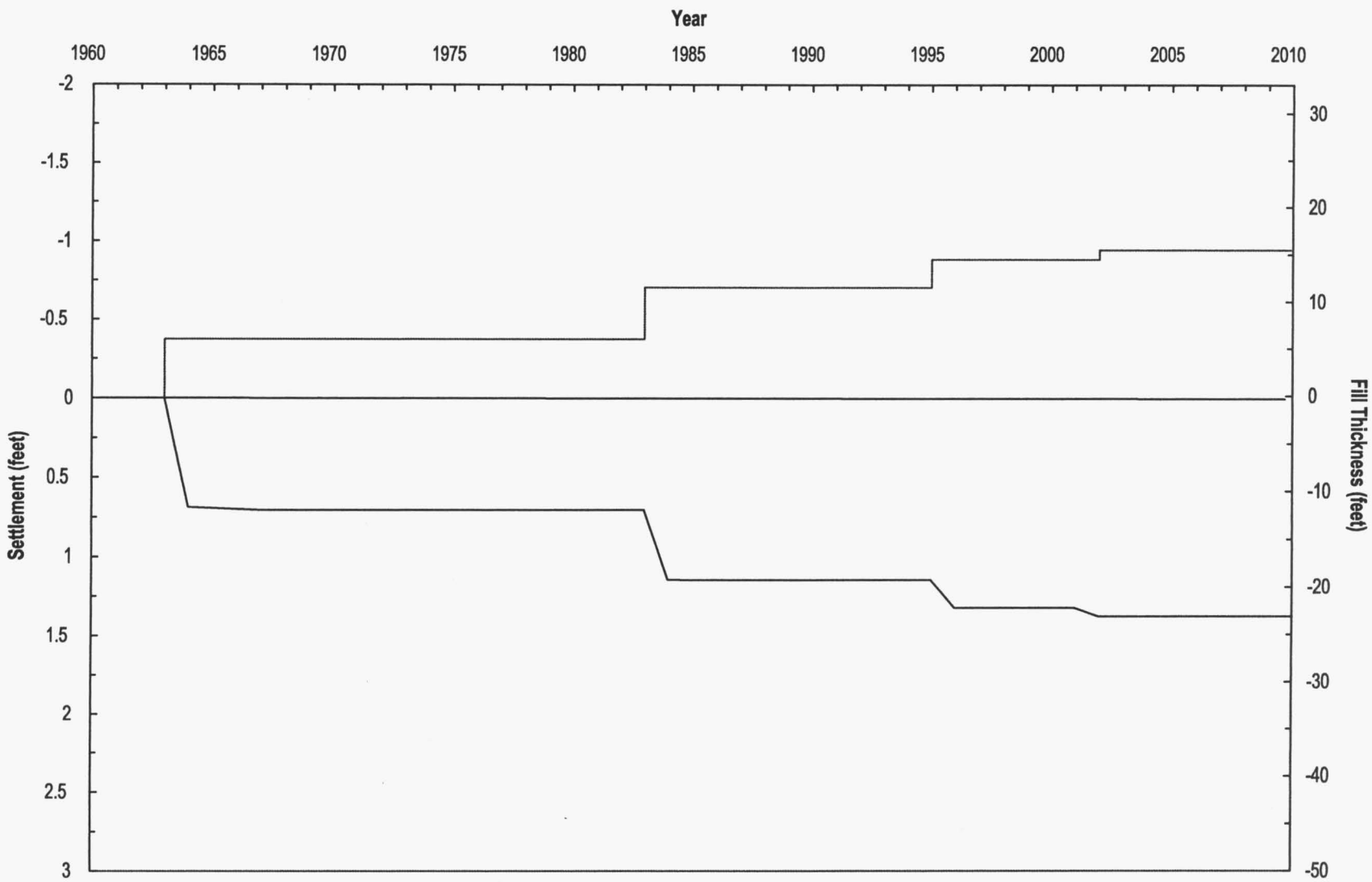
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 18	Figure 65
7/15/02			
URS			

TEMP002



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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 19	Figure 66
7/15/02			
URS			

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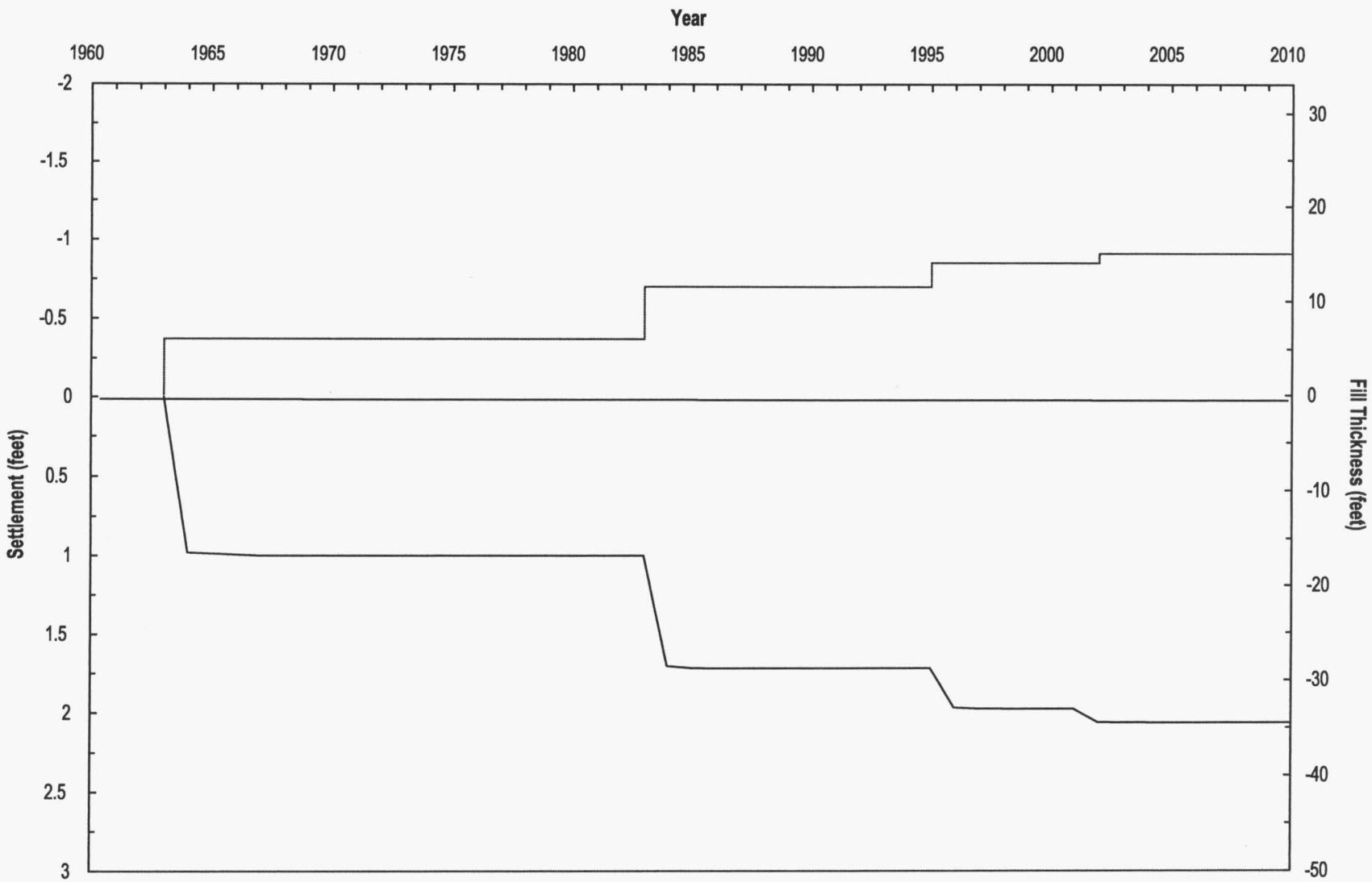

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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 20	Figure 67
7/15/02			
URS			

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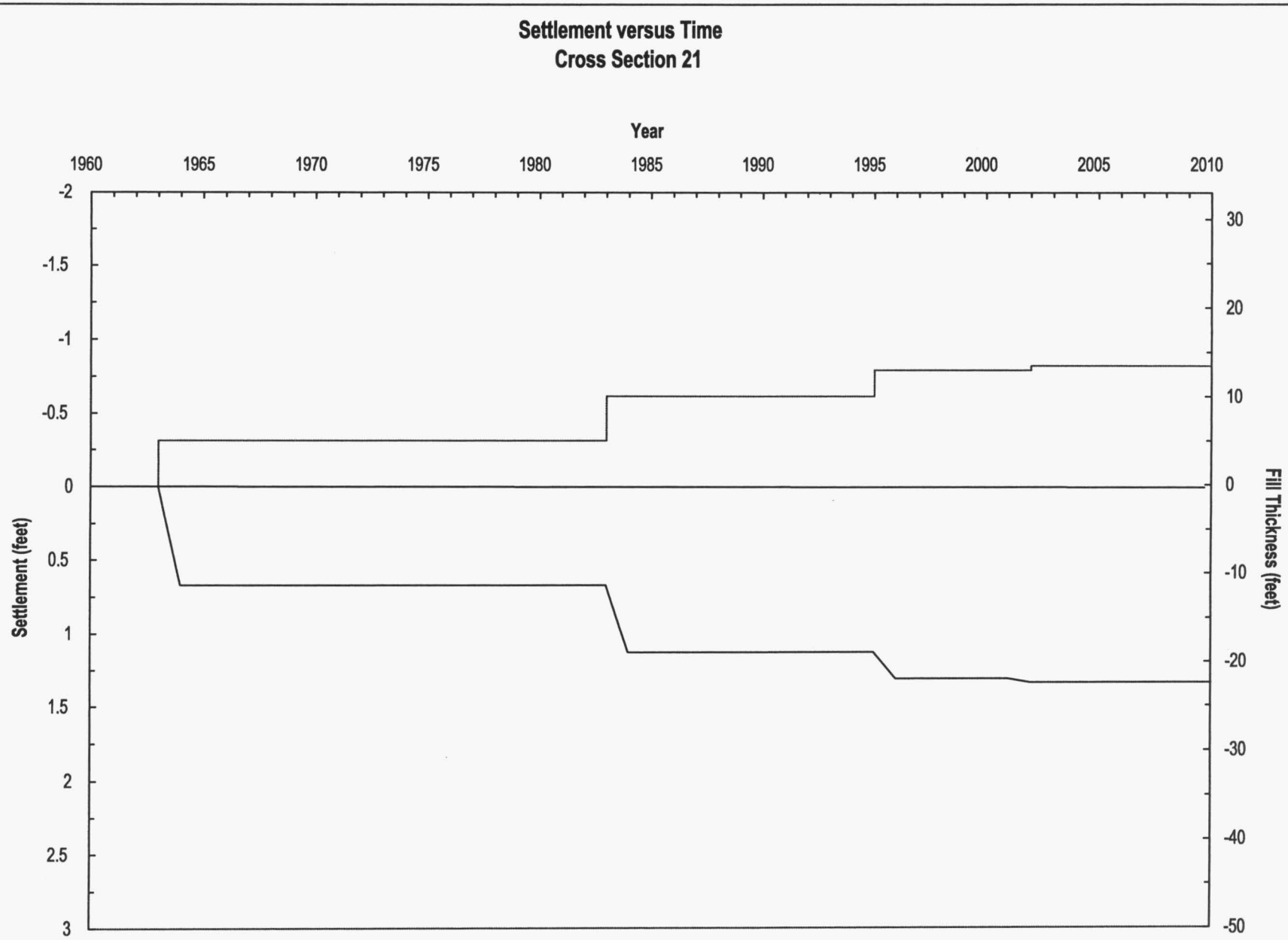

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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 21	Figure 68
7/15/02			
URS			

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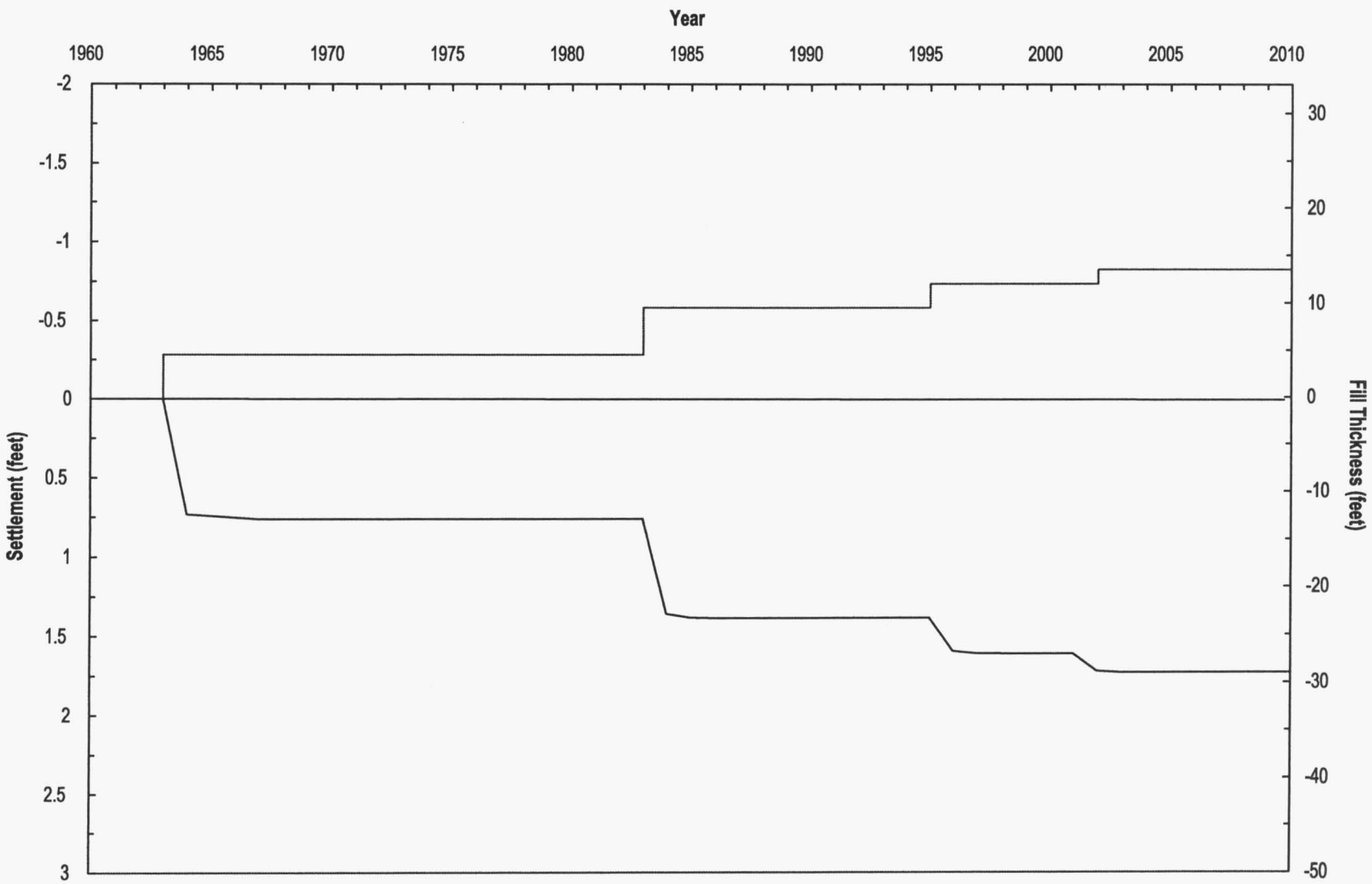

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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 22	Figure 69
7/15/02			
URS			

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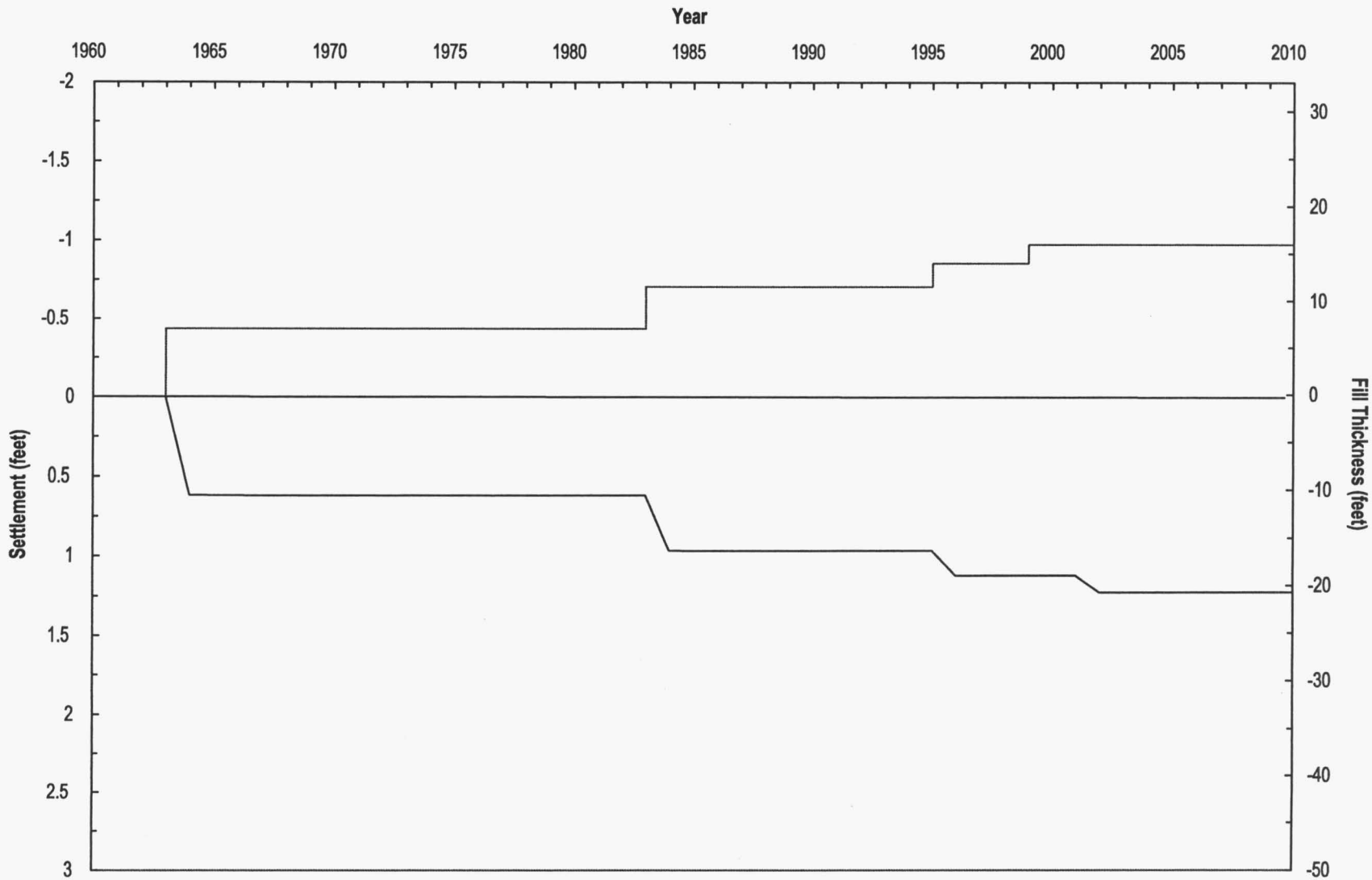

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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 23	Figure 70
7/15/02			
URS			

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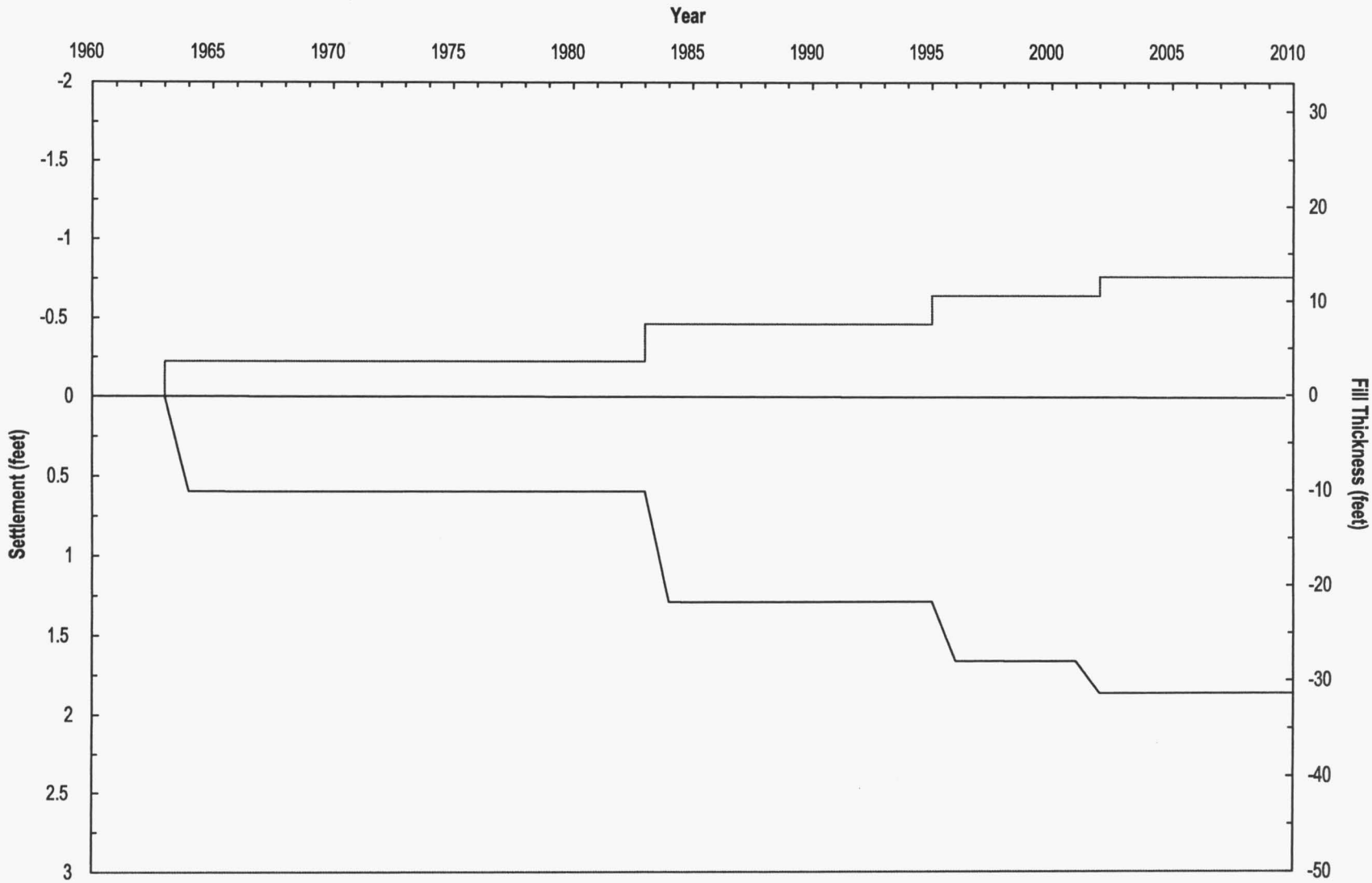

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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 24	Figure 71
7/15/02			
URS			

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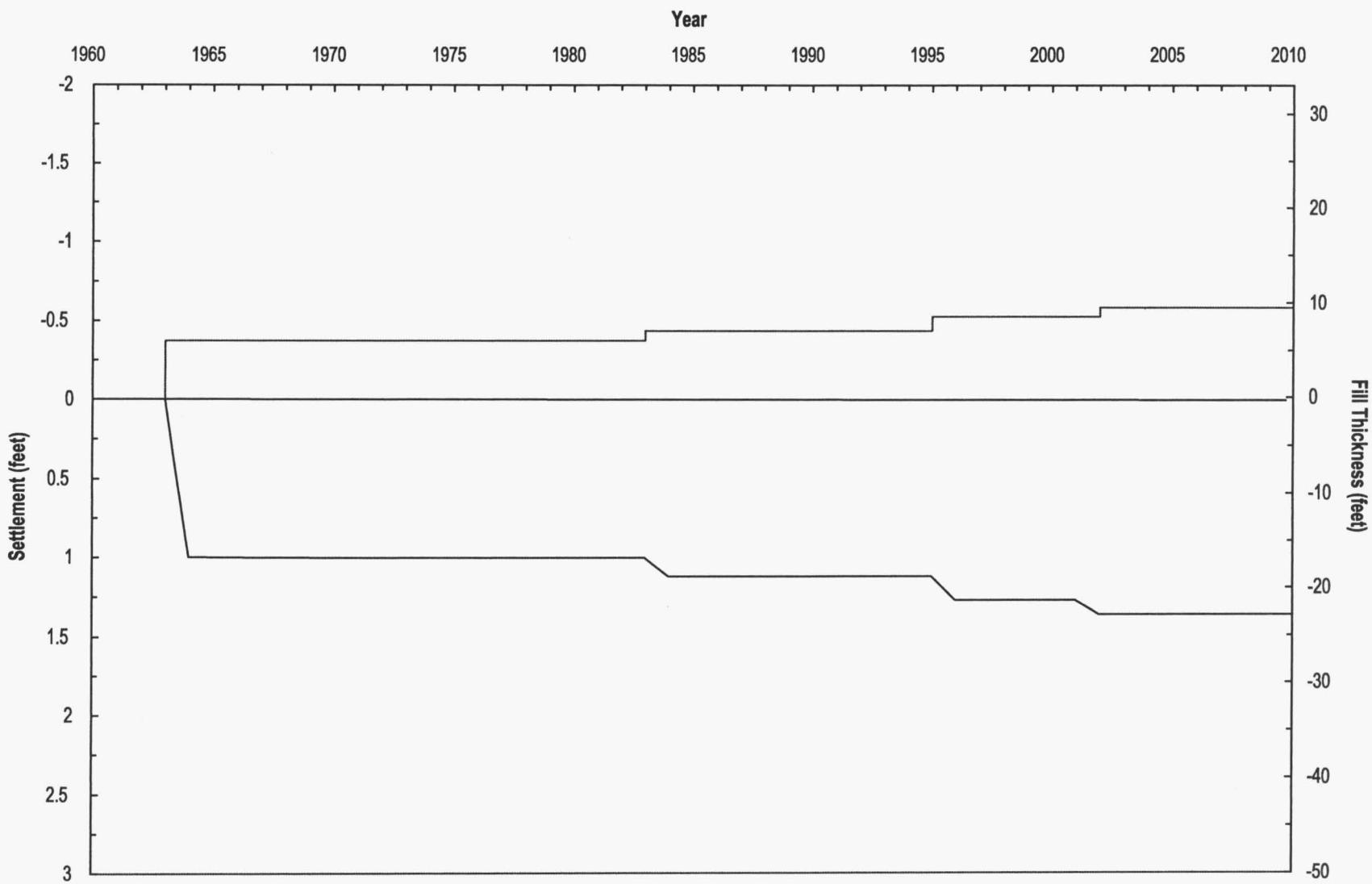

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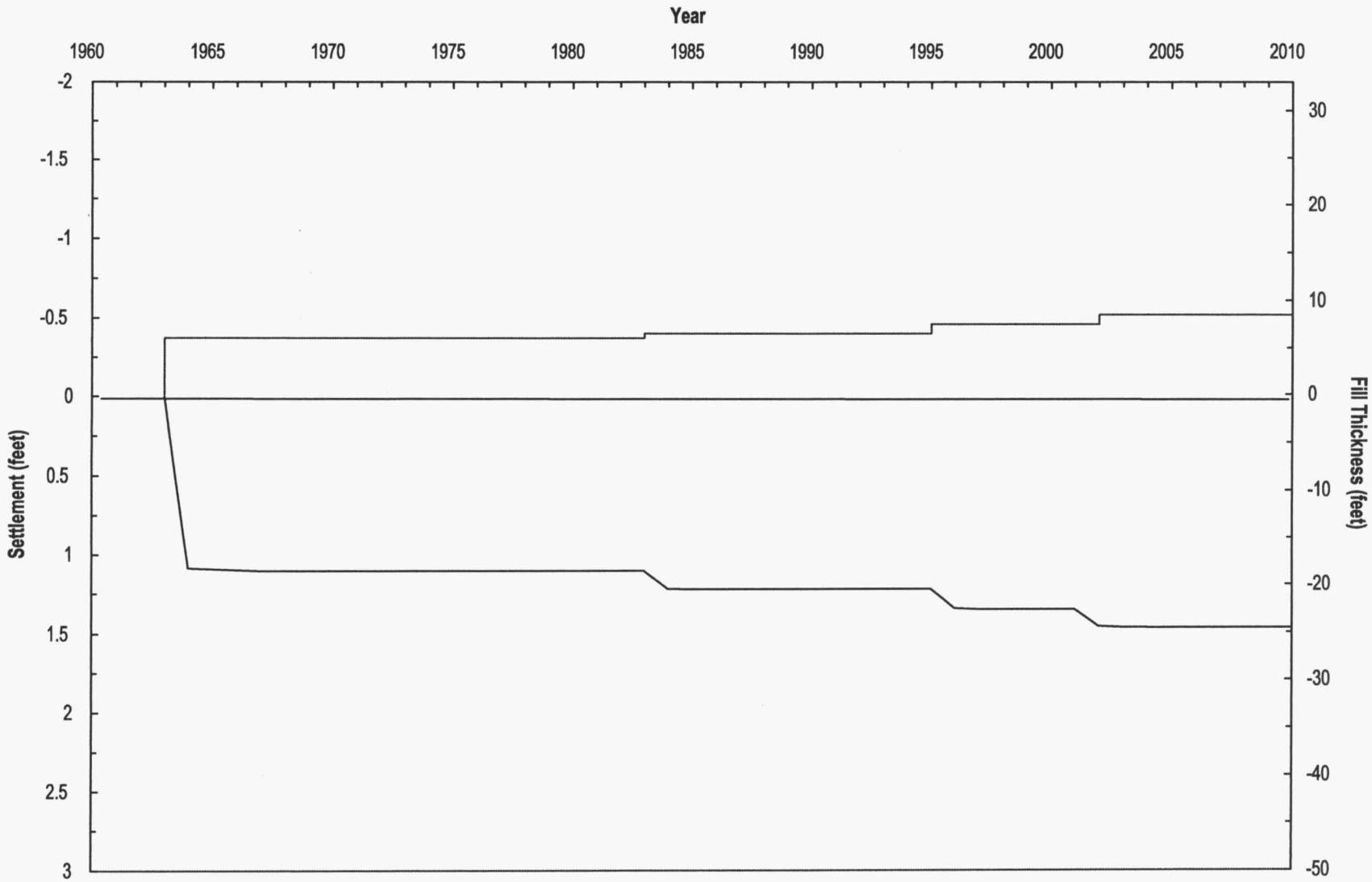
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	Estimated Time Rate of Settlement for Cross Section 25	Figure 72
7/15/02	Santa Clara County, California		
URS			

TEMP002



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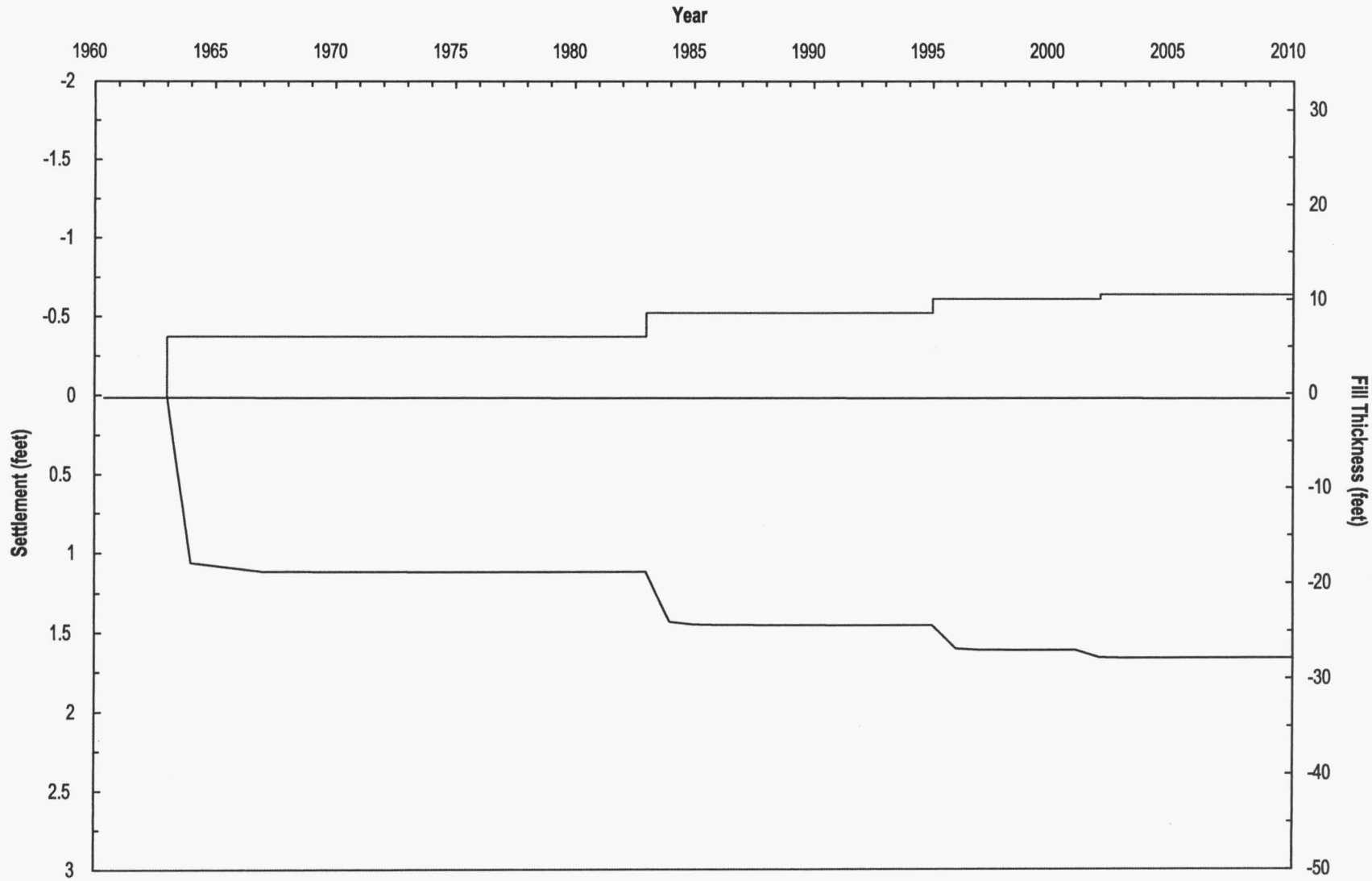
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 26	Figure 73
7/15/02			
URS			

TEMP002



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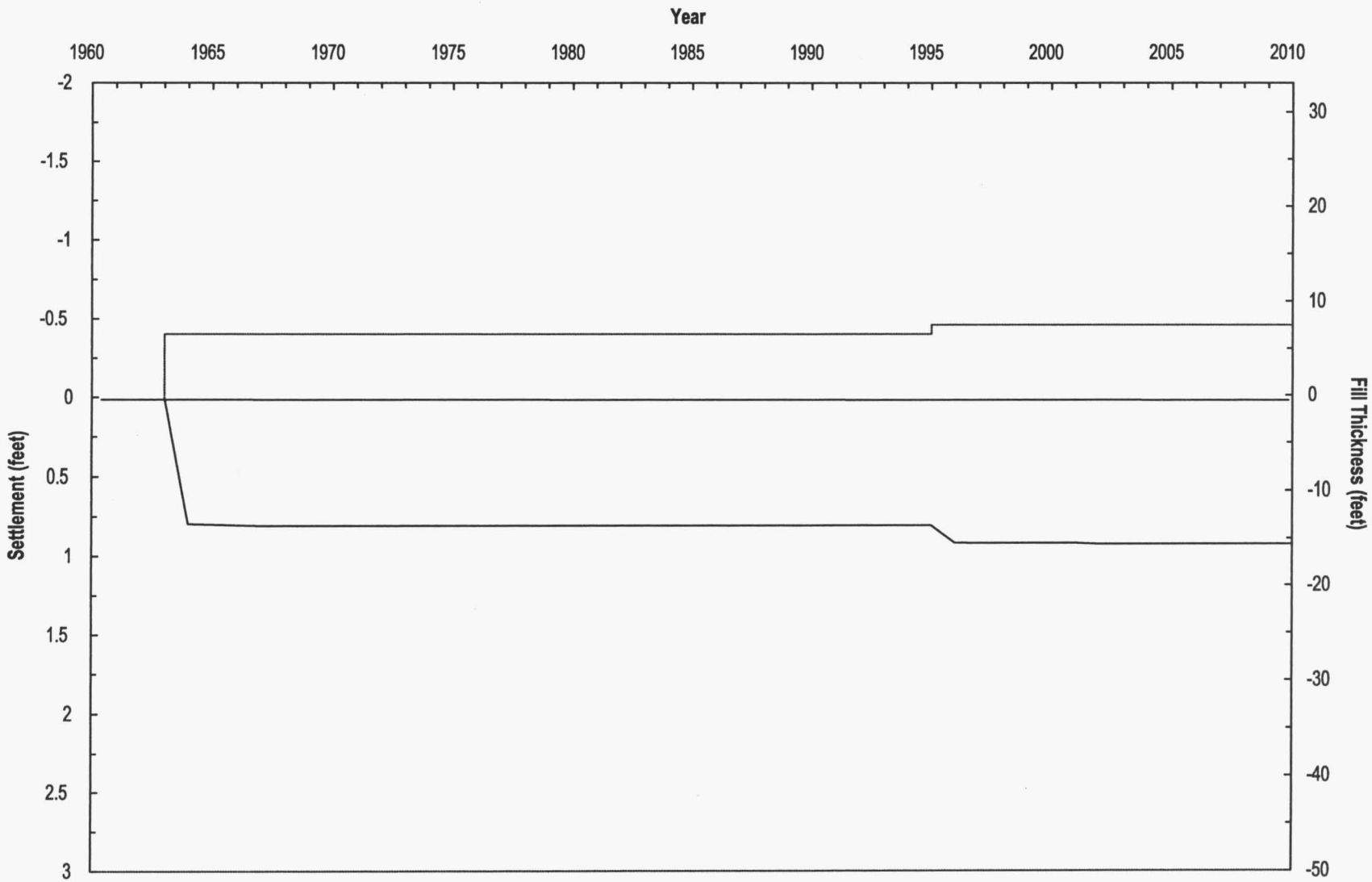
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	Estimated Time Rate of Settlement for Cross Section 27	Figure 74
7/15/02	Santa Clara County, California		
URS			

TEMPO02



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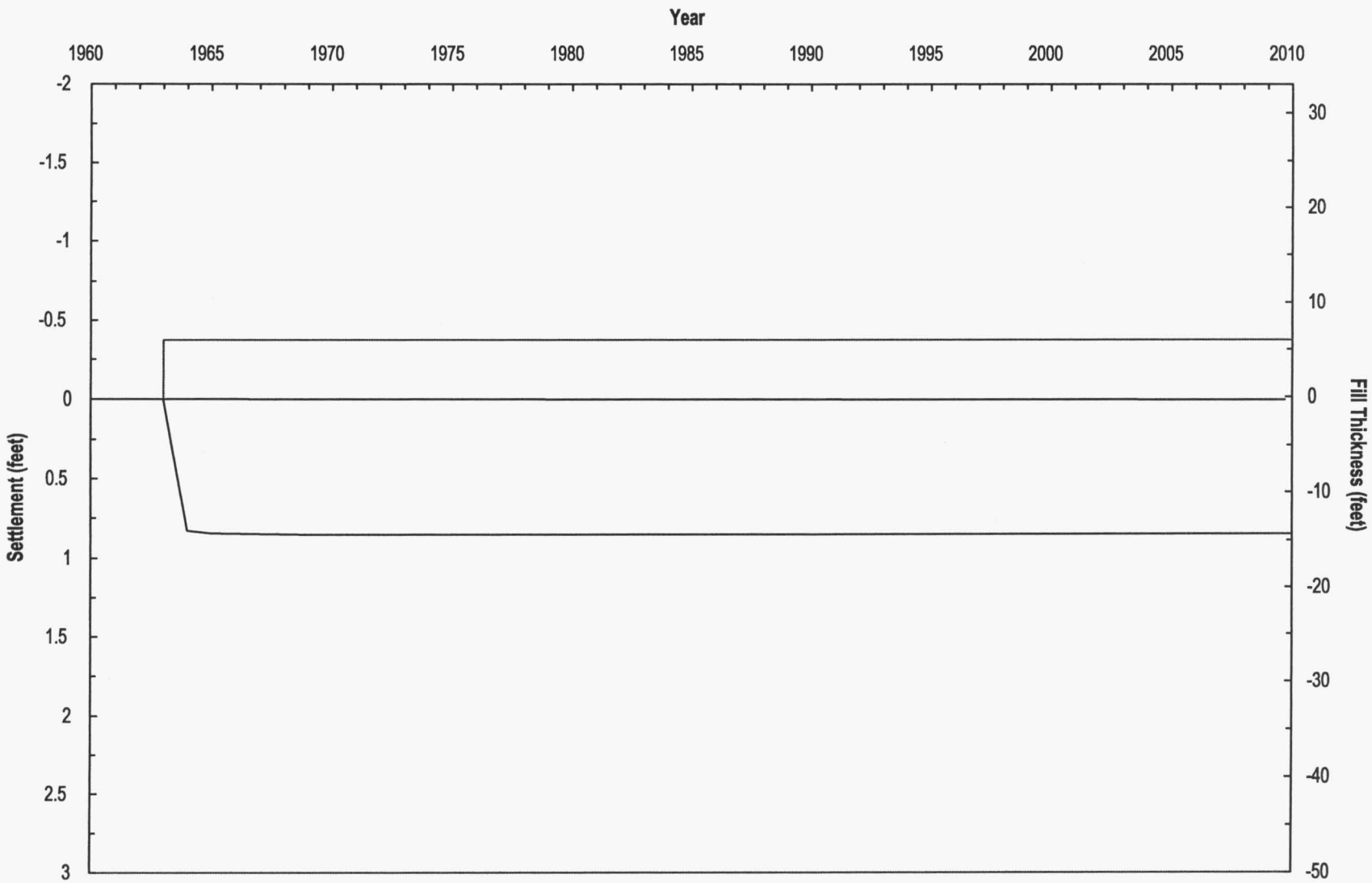
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 28	Figure 75
7/15/02			
URS			

TEMP002



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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 29	Figure 76
7/15/02			
URS			

TEMP002

Coefficient of Permeability k in cm per sec (log scale)

10^2 | 10^1 | 1.0 | 10^{-1} | 10^{-2} | 10^{-3} | 10^{-4} | 10^{-5} | 10^{-6} | 10^{-7} | 10^{-8} | 10^{-9}

Drainage	Good		Poor	Practically Impervious
Soil types	Clean gravel	Clean sands, clean sand and gravel mixtures	Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.	"Impervious" soils, e.g., homogeneous clays below zone of weathering
			"Impervious" soils modified by effects of vegetation and weathering	
Direct determination of k	Direct testing of soil in its original position—pumping tests. Reliable if properly conducted. Considerable experience required			
	Constant-head permeameter. Little experience required			
Indirect determination of k		Falling-head permeameter. Reliable. Little experience required	Falling-head permeameter. Unreliable. Much experience required	Falling-head permeameter. Fairly reliable. Considerable experience necessary
	Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels			Computation based on results of consolidation tests. Reliable. Considerable experience required

After A. Casagrande and R. E. Fadum

FIG. 2.8 Permeability and drainage characteristics of soils. (Table 6, p. 48, of *Soil Mechanics in Engineering Practice*, by K. Terzaghi and R. B. Peck, John Wiley and Sons, New York, 1948.)

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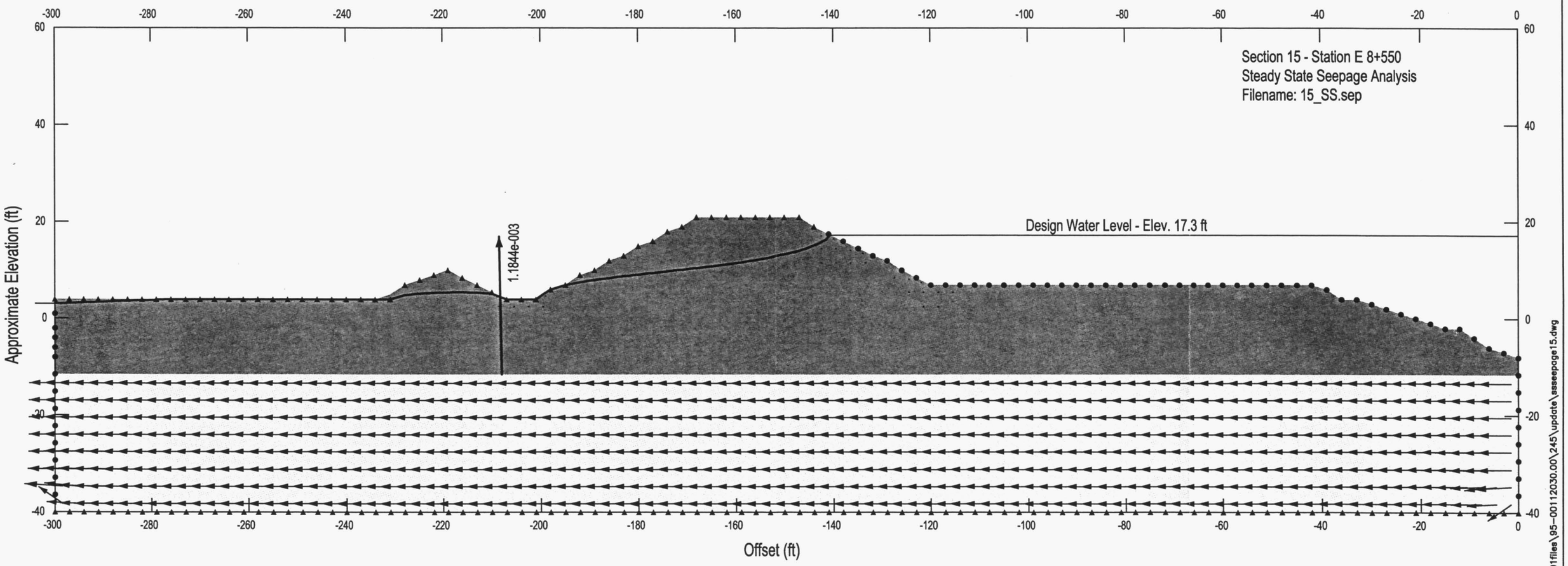
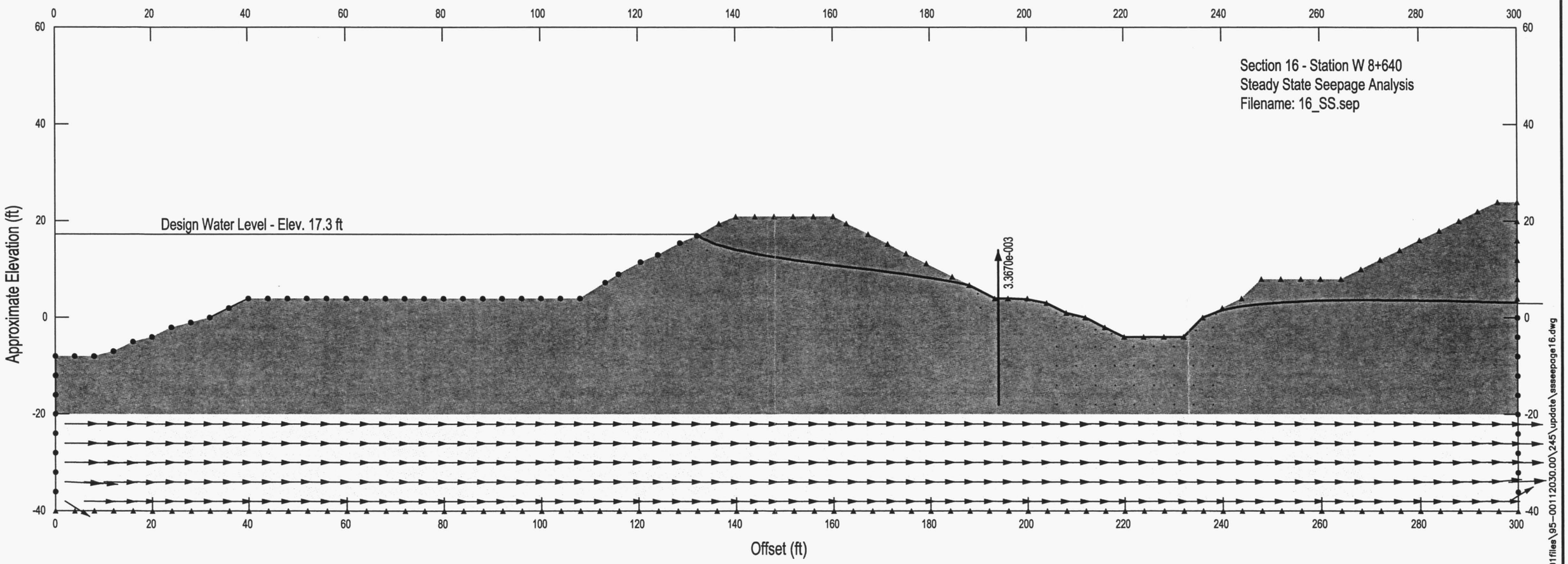
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	PERMEABILITY AND DRAINAGE CHARACTERISTICS OF SOILS	Figure 77
7/15/02			
URS			

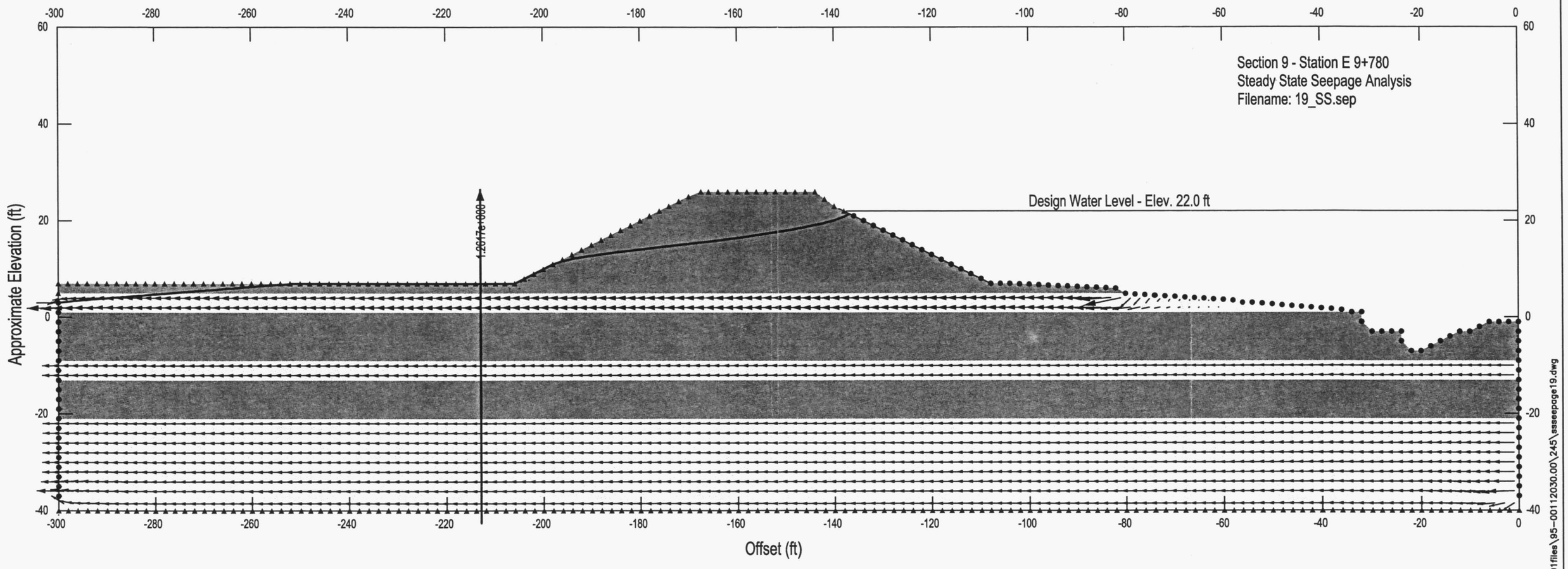
TEMP001



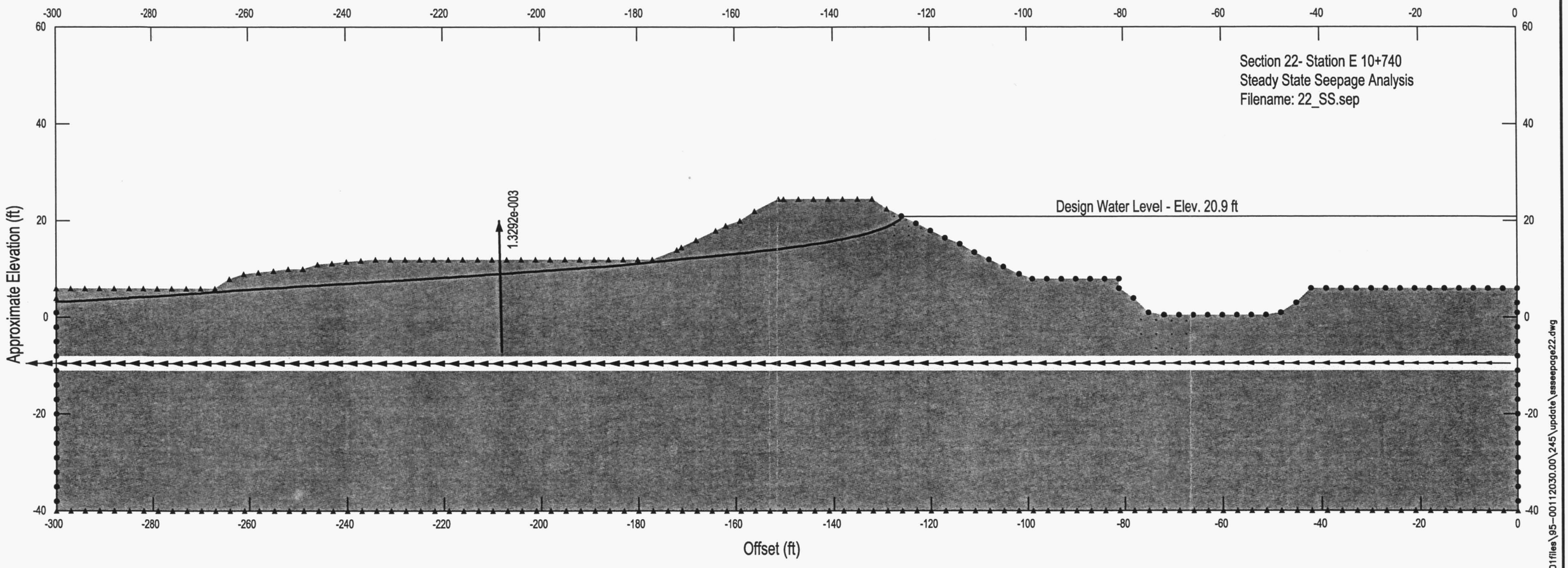
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 15	Figure 78
TEMPO04 7/15/02	Santa Clara County, California		
	URS		



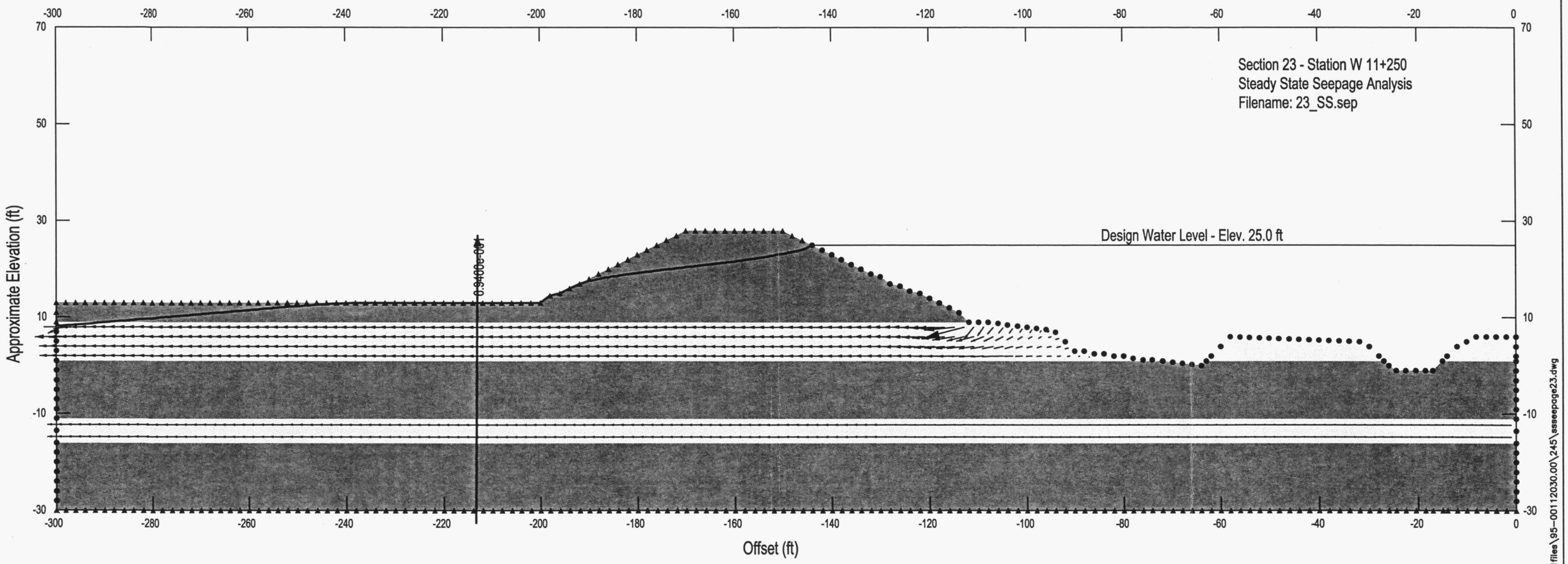
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 16	Figure 79
TB&P004 7/15/02	URS		



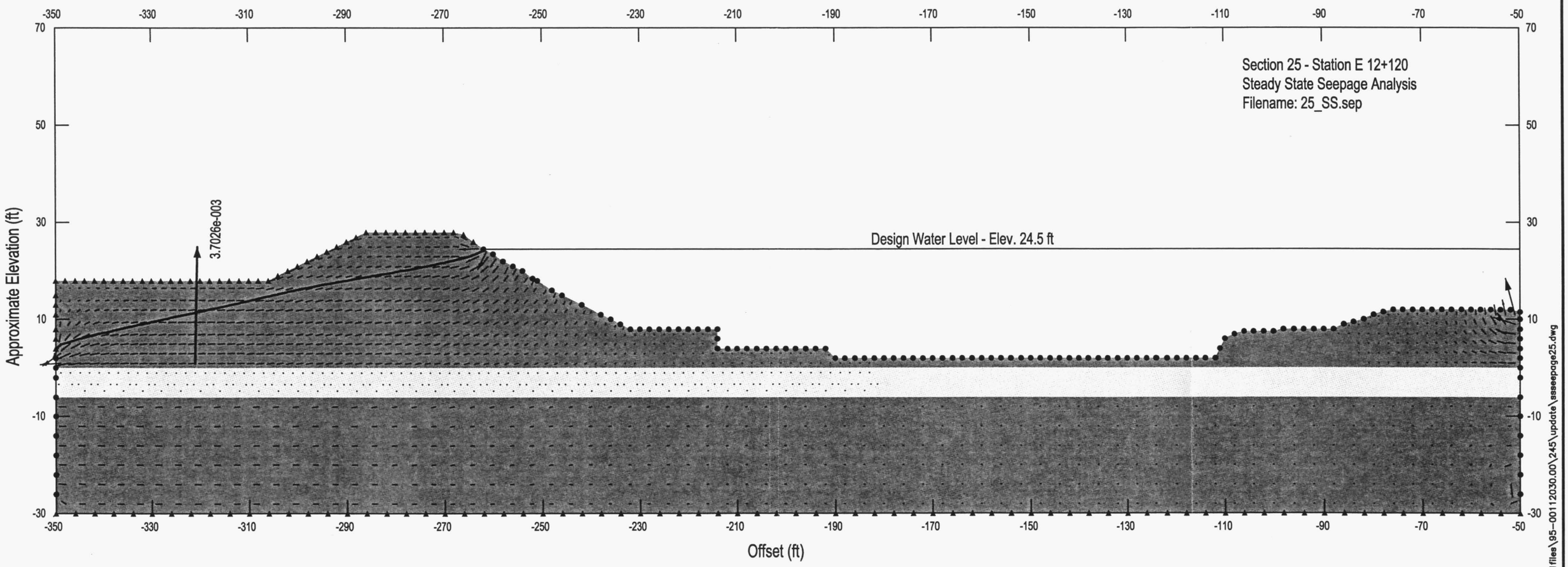
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 19	Figure 80
TEMP004 7/15/02	URS		



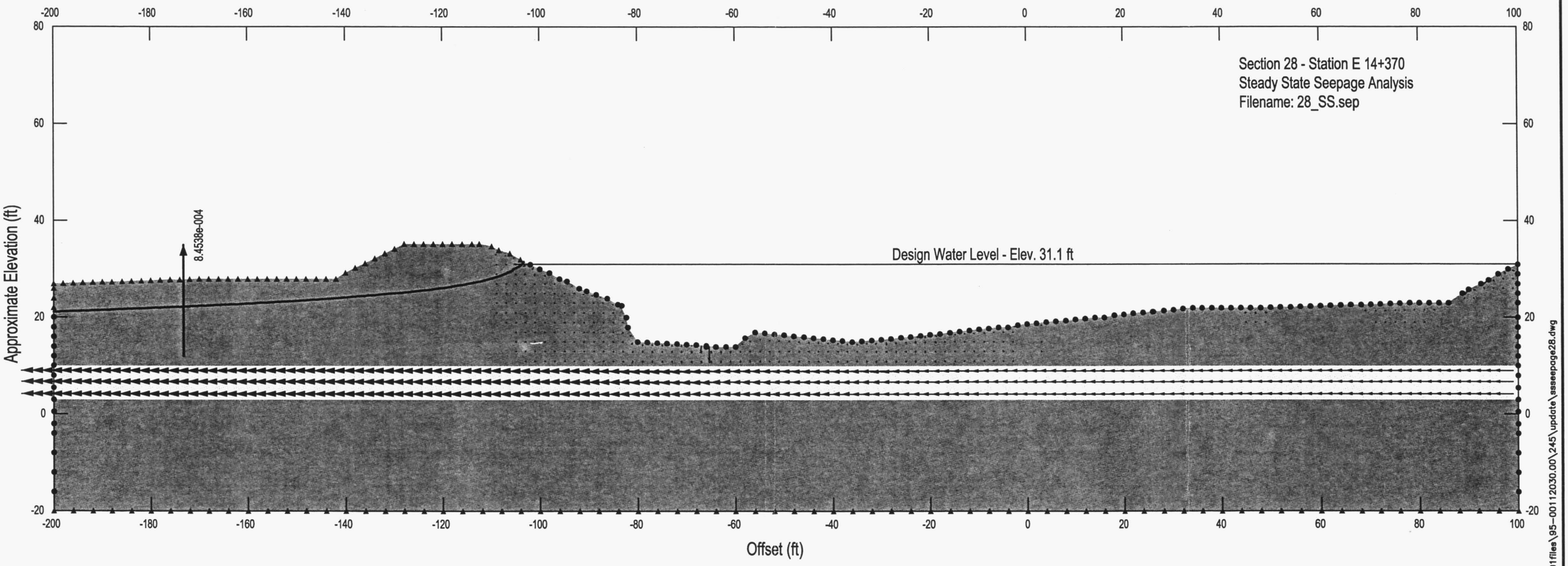
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 22	Figure 81
TEMPFOOT 7/15/02	Santa Clara County, California		
	URS		



Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 23	Figure 82
TEMPFOOH 7/15/02	Santa Clara County, California		
	URS		



Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 25	Figure 83
TEMP004 7/15/02	URS		



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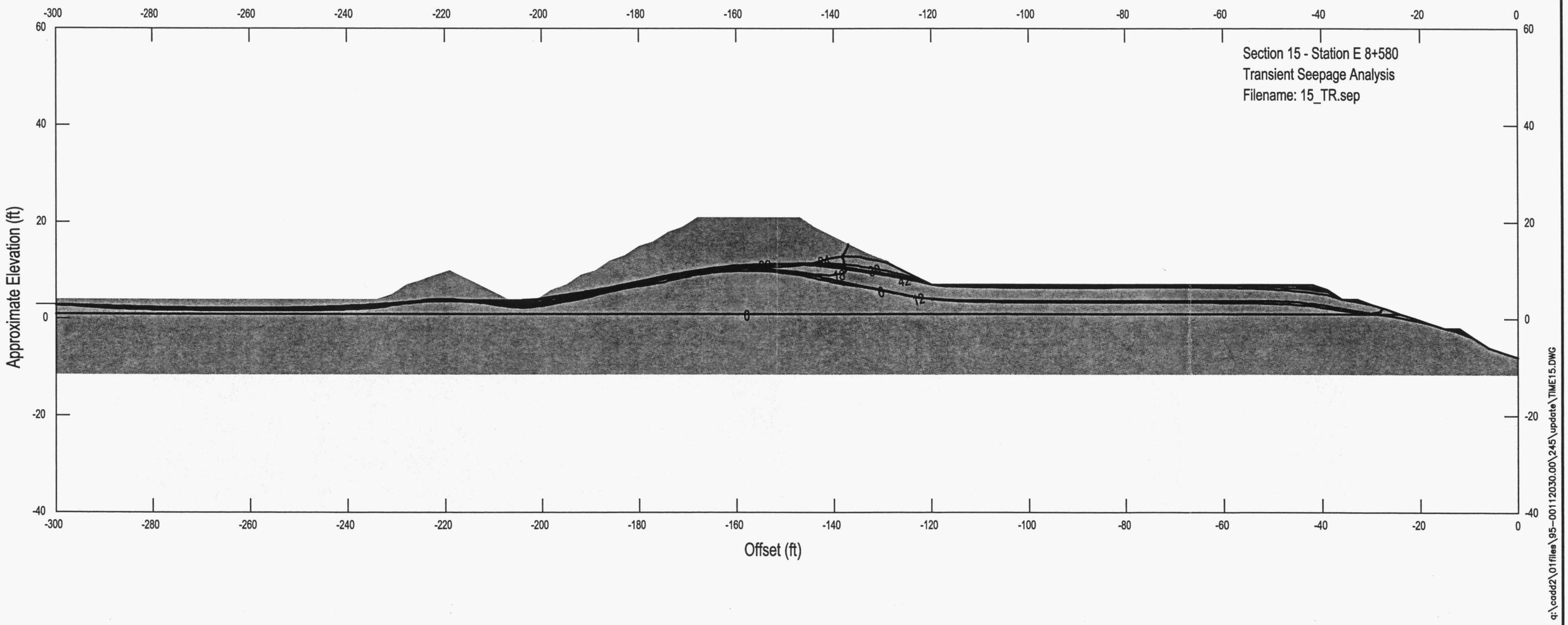
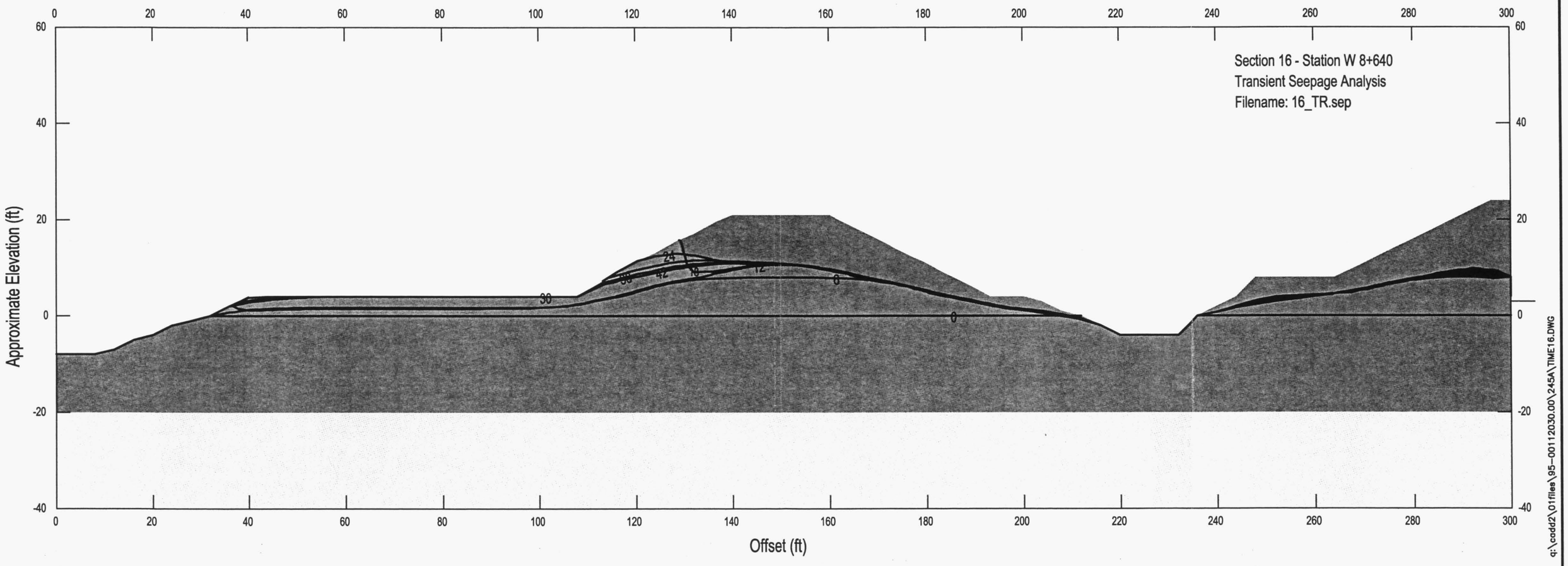
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 28	Figure 84
TEMPO04 7/15/02	Santa Clara County, California		
	URS		



LEGEND:

36 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

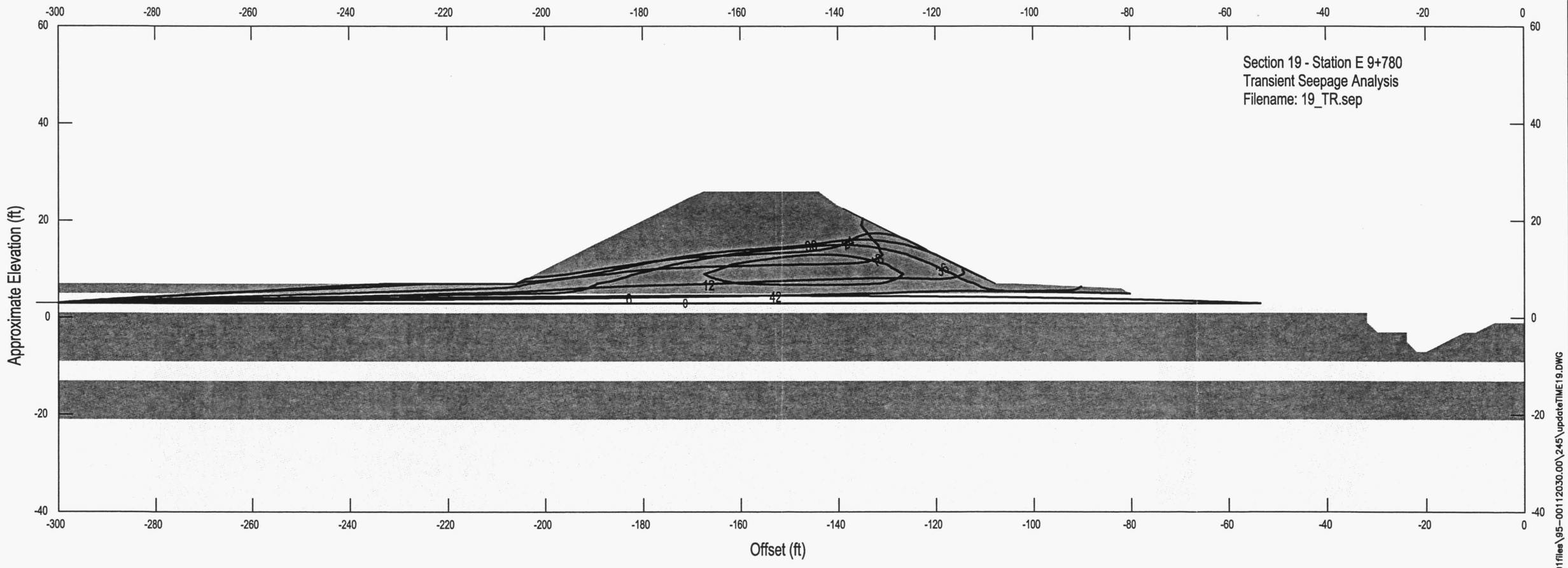
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 15	Figure 85
7/15/02	URS		



LEGEND:

36 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

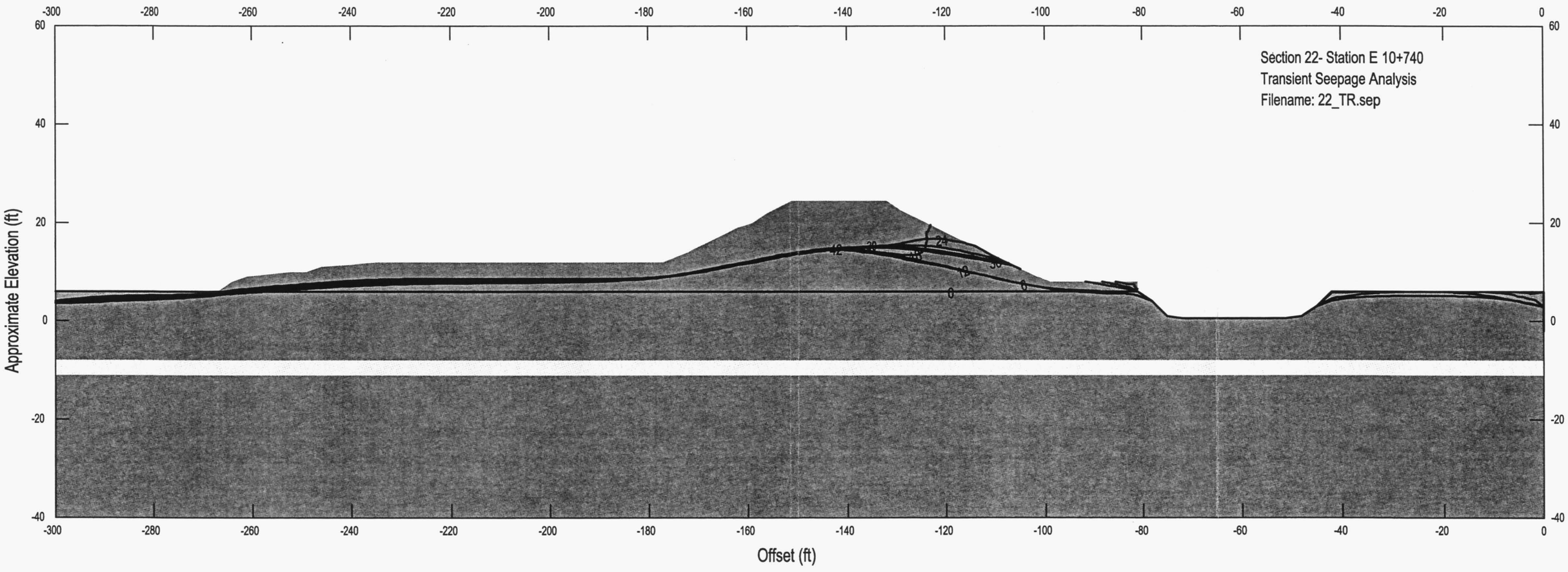
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 16	Figure 86
7/15/02	URS		



LEGEND:

30 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 19	Figure 87
7/15/02	URS		



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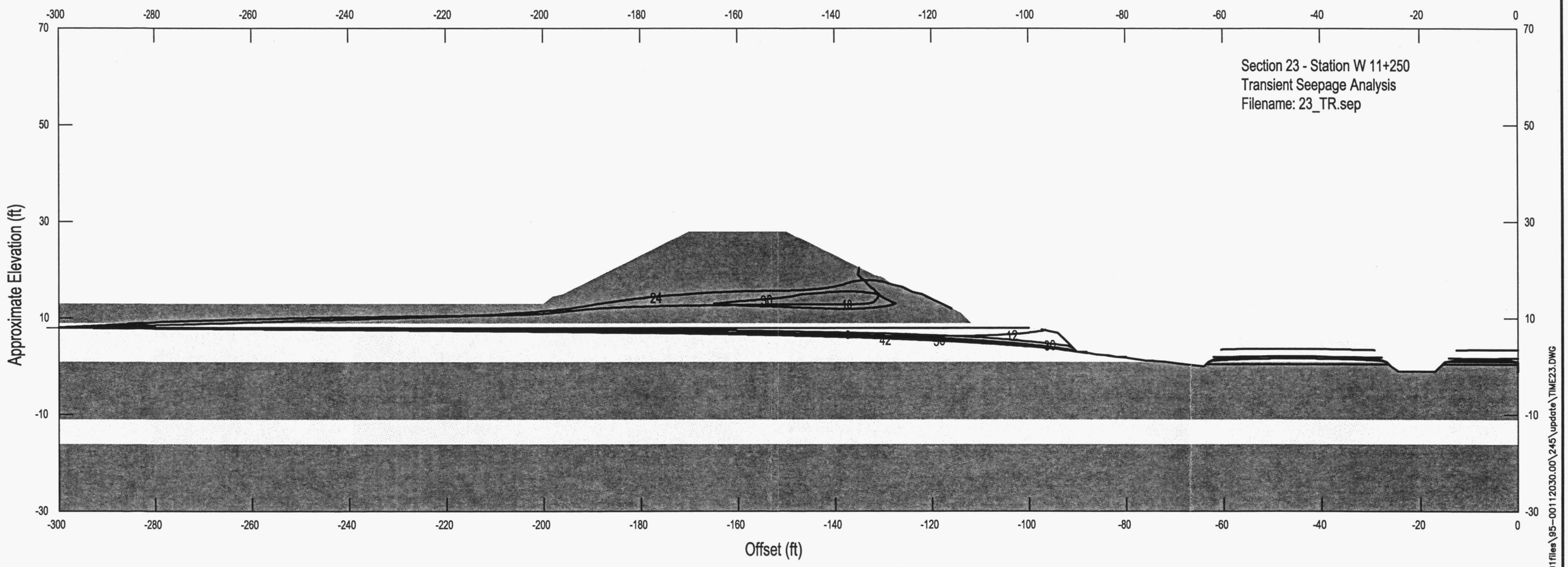
LEGEND:

30 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

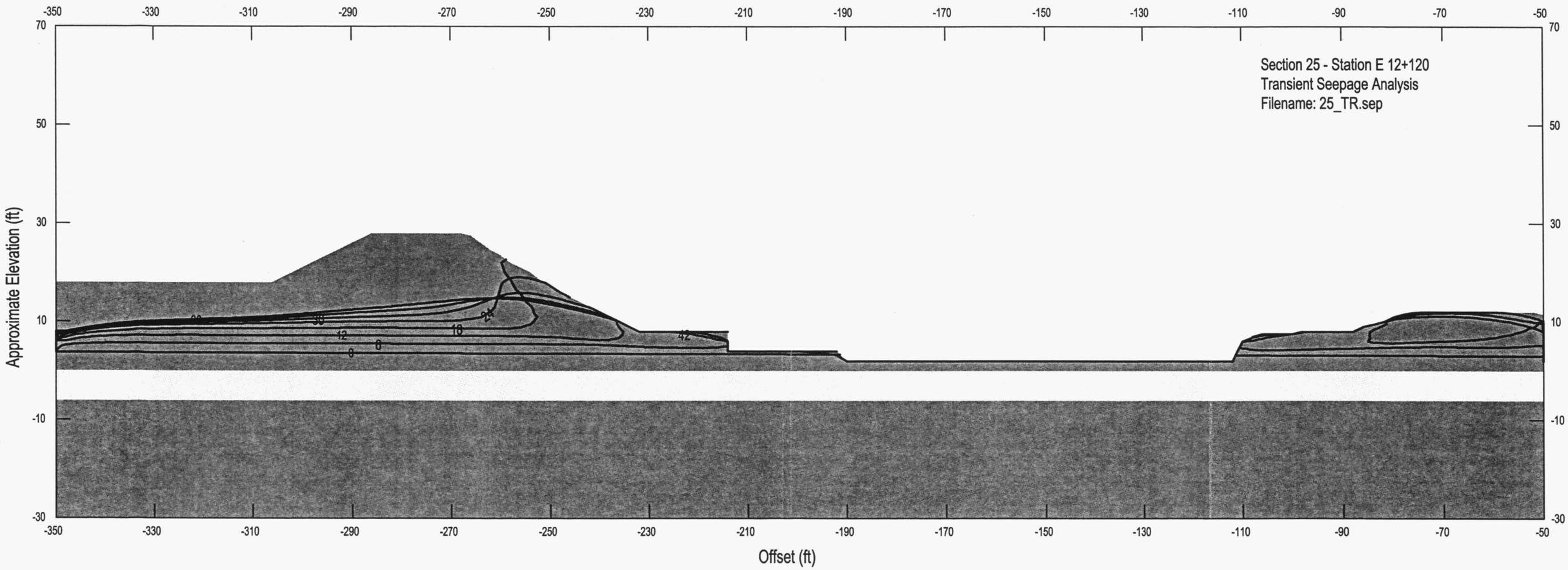
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 22	Figure 88
7/15/02	URS		



LEGEND:

30 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 23	Figure 89
7/15/02	URS		



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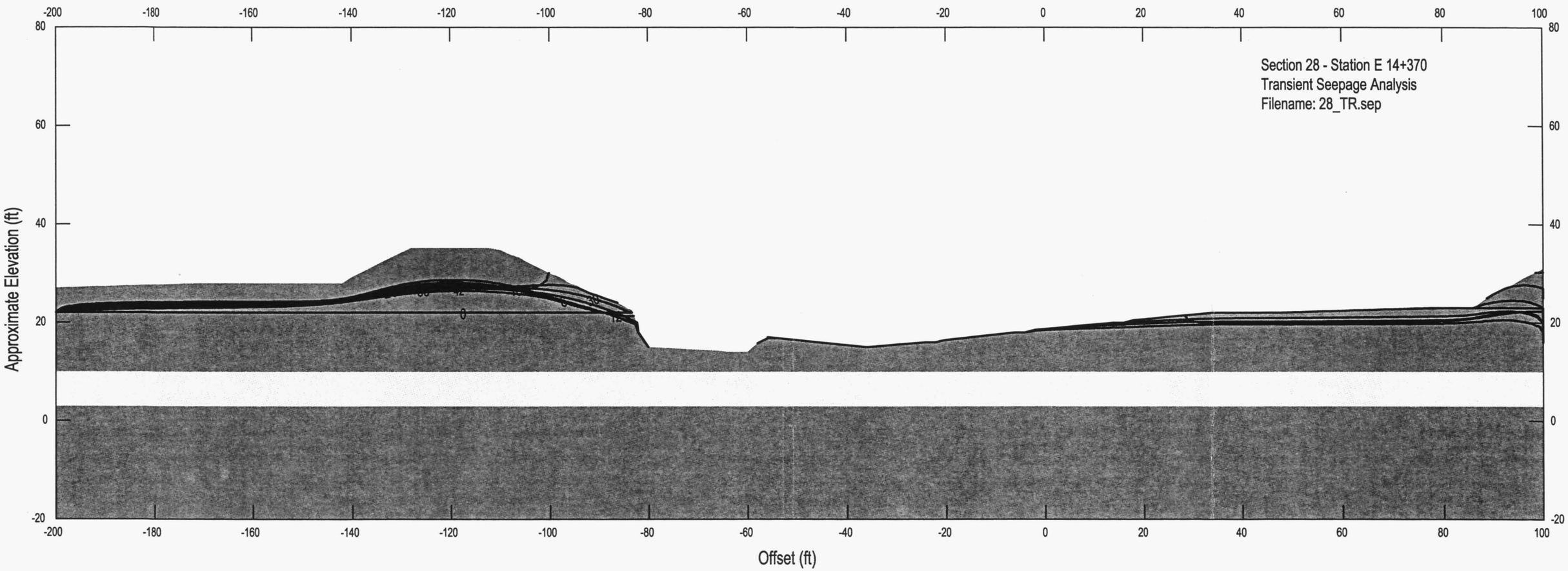
LEGEND:

30 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 25	Figure 90
7/15/02	URS		



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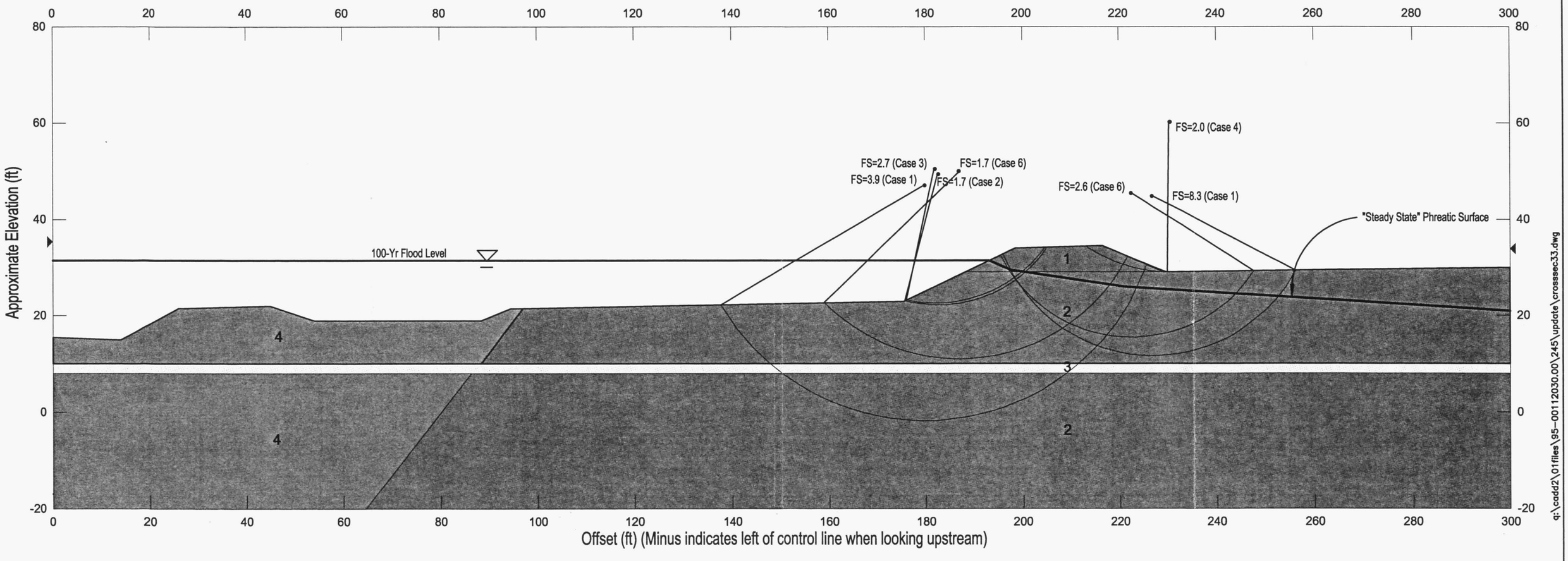
LEGEND:

30 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 28	Figure 91
7/15/02	URS		

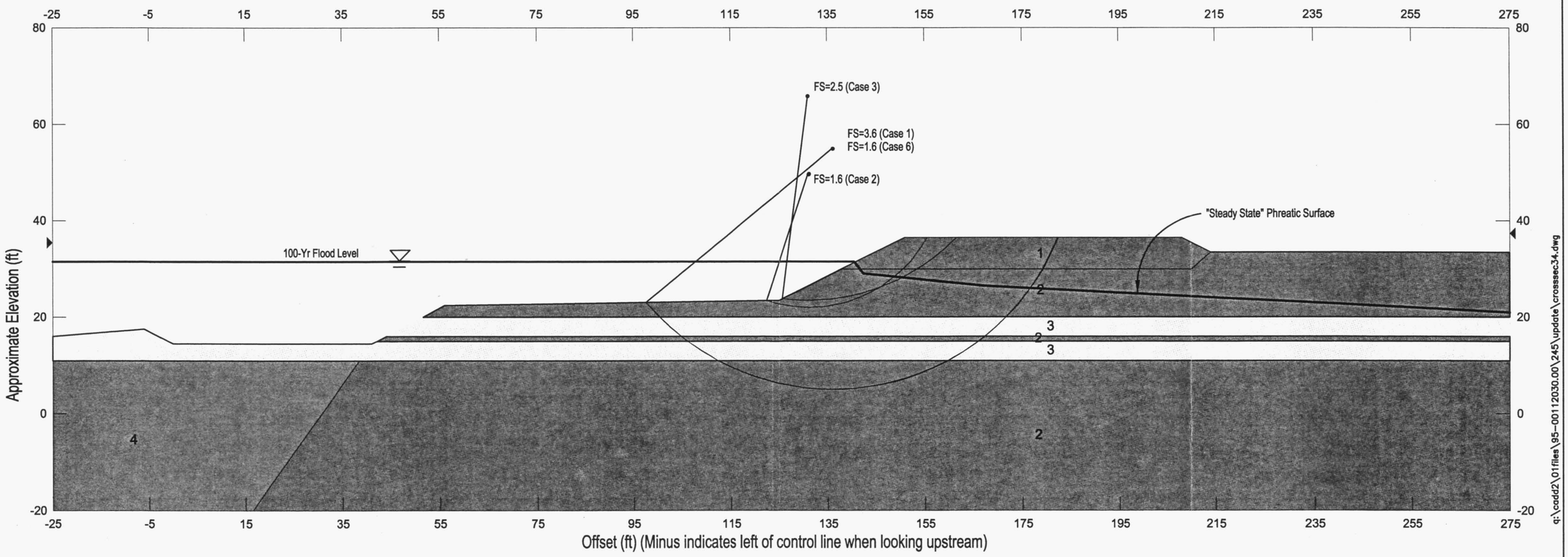
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 94	116 116	19 19	138 138	33 33	0 0	3000 2040	Selected
2	Lean CLAY	Table 6 Figure 94	100 100	24 24	124 124	30 30	150 150	1400 1400	Selected
3	SAND				135	35 35	0 0		Estimated from N = 25, σ_v' = 3.1 ksf, Dr = 70% Selected
4	Lean CLAY (Channel Sediments)	Table 6	97	27	123 123	30 30	150 150	750	Selected



Project No. 95-00112030.00 TEMPOO4 7/15/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California URS	STABILITY ANALYSIS CROSS SECTION 33 STATION 13+890 WEST (SETBACK) LEVEE

Figure 92

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 6 Figure 95	116 116	19 19	138 138	33 33	0 0	3000 2040	Selected
2	Lean CLAY	Table 6 Figure 95	100 100	24 24	124 124	30 30	150 150	1400 1400	Selected
3	SAND				135	35 35	0 0		Estimated from N = 25, σ_v' = 3.1 ksf, Dr = 70% Selected
4	Lean CLAY (Channel Sediments)	Table 6	97	27	123 123	30 30	150 150	750	Selected



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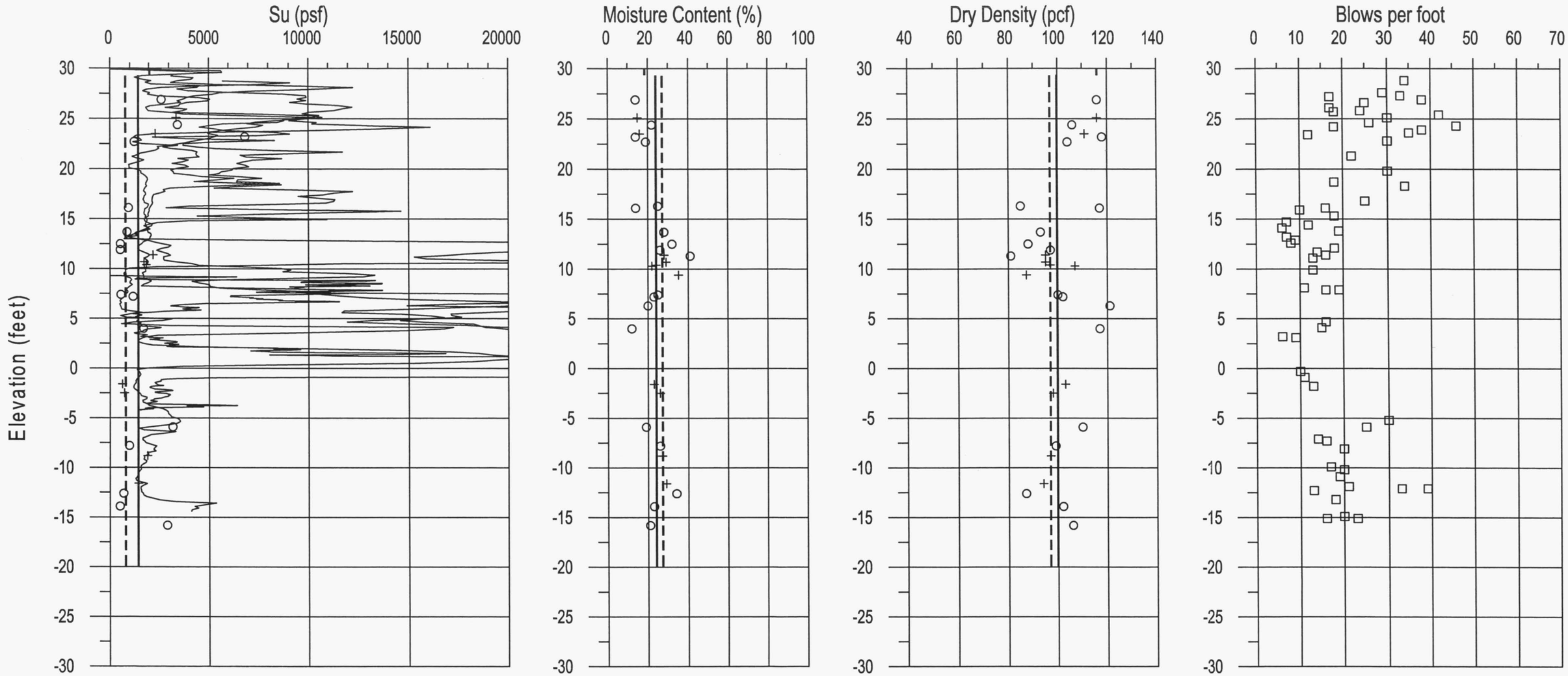
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 34 STATION 14+430 WEST (SETBACK) LEVEE	Figure 93
TEMP004 7/15/02	URS		



Data from the following borings and CPTs:

SPRR-3	EB-13	WB-12	EC-10	WC-07
SPRR-4	EB-14	WB-13	EC-11	WC-08
SPRR-5	WB-10	CB-07	EC-12	
SPRR-6	WB-11	CB-08	WC-06	

Levee Fill and Foundation Soil

+ Fat Clay

○ Lean Clay

□ All Soil Types

Channel Sediments

+ Fat Clay

● Lean Clay

■ All Soil Types

LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

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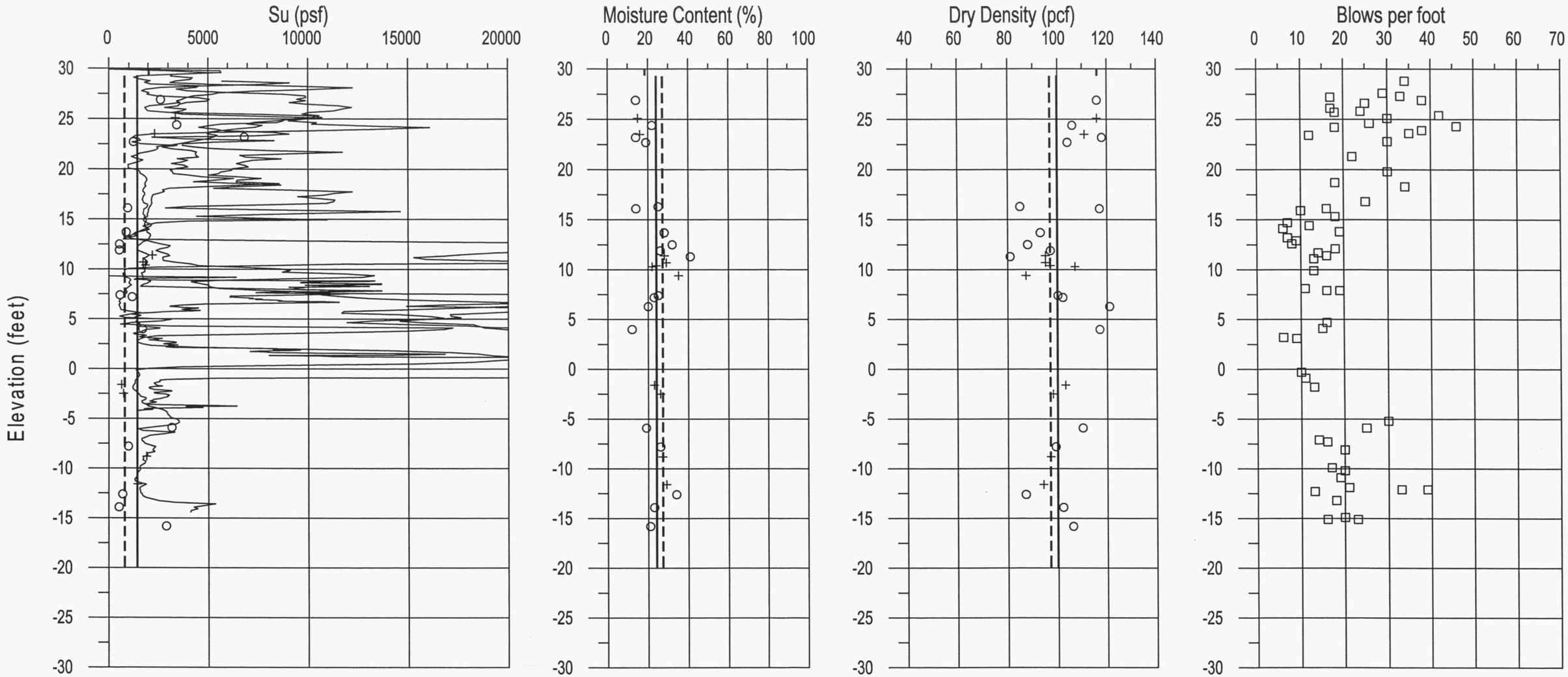
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 33	Figure 94
7/15/02	URS		



Data from the following borings and CPTs:

SPRR-3	EB-13	WB-12	EC-10	WC-07
SPRR-4	EB-14	WB-13	EC-11	WC-08
SPRR-5	WB-10	CB-07	EC-12	
SPRR-6	WB-11	CB-08	WC-06	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

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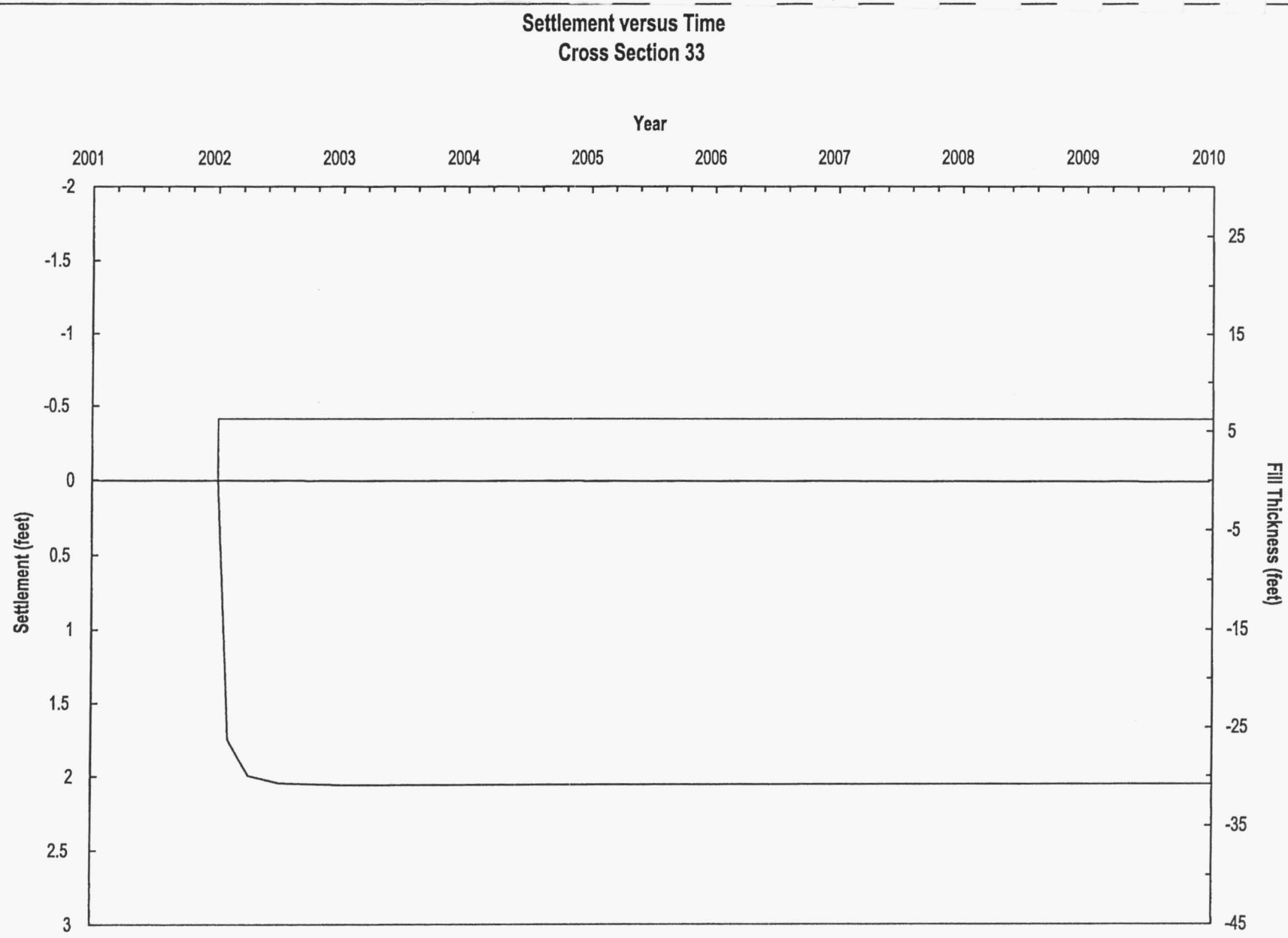
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 34	Figure 95
7/15/02	URS		



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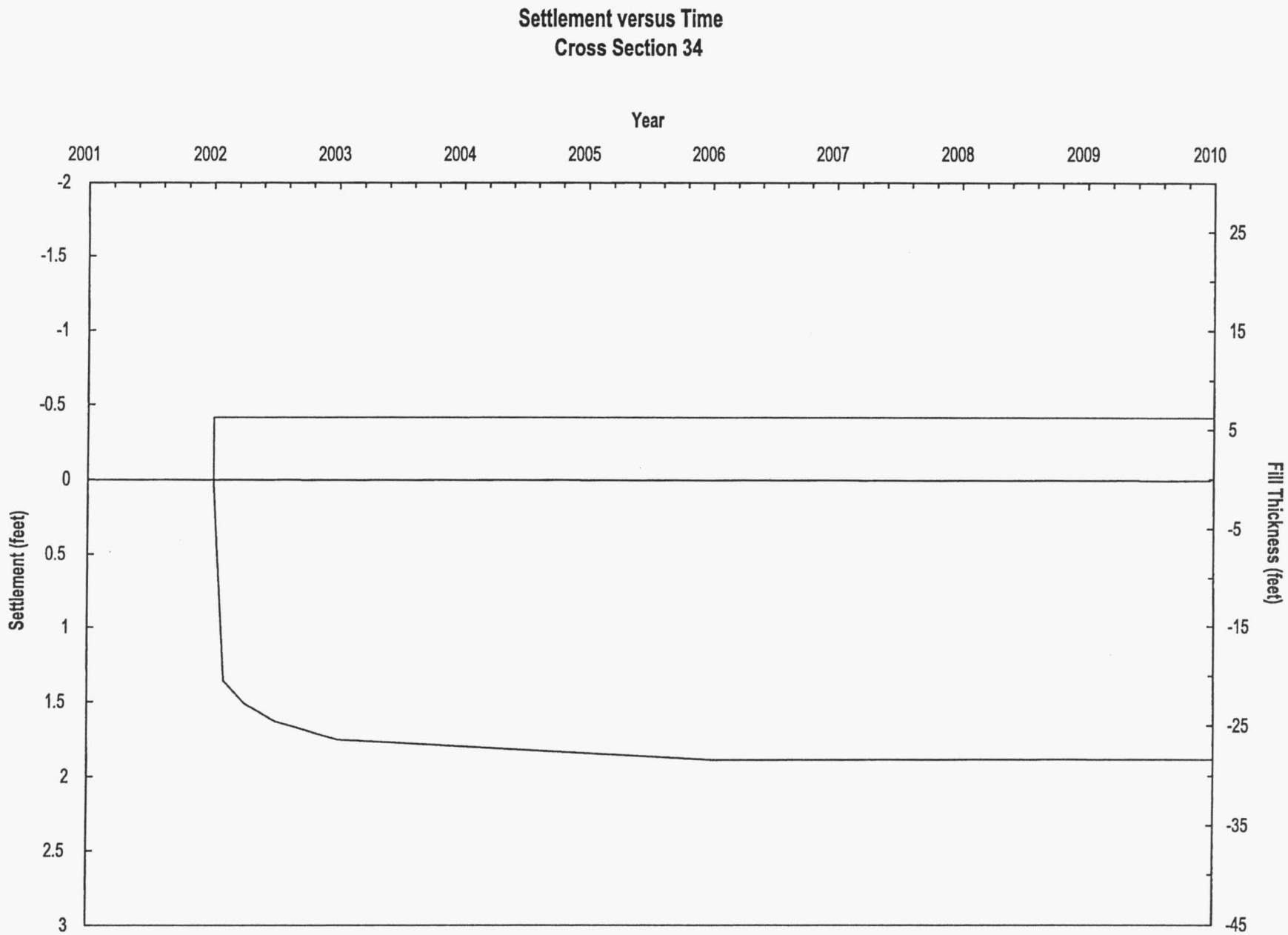
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	Estimated Time Rate of Settlement for Cross Section 33	Figure 96
7/15/02	Santa Clara County, California		
URS			



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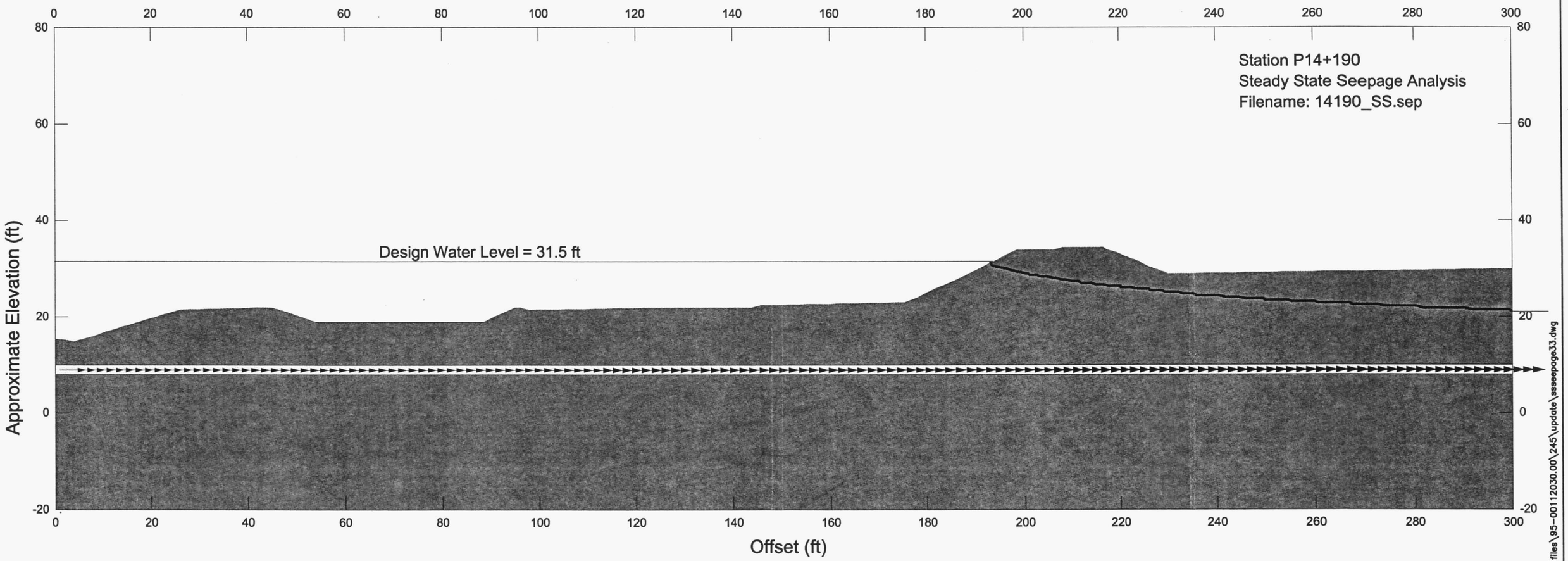
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	Estimated Time Rate of Settlement for Cross Section 34	Figure 97
7/15/02			
URS			



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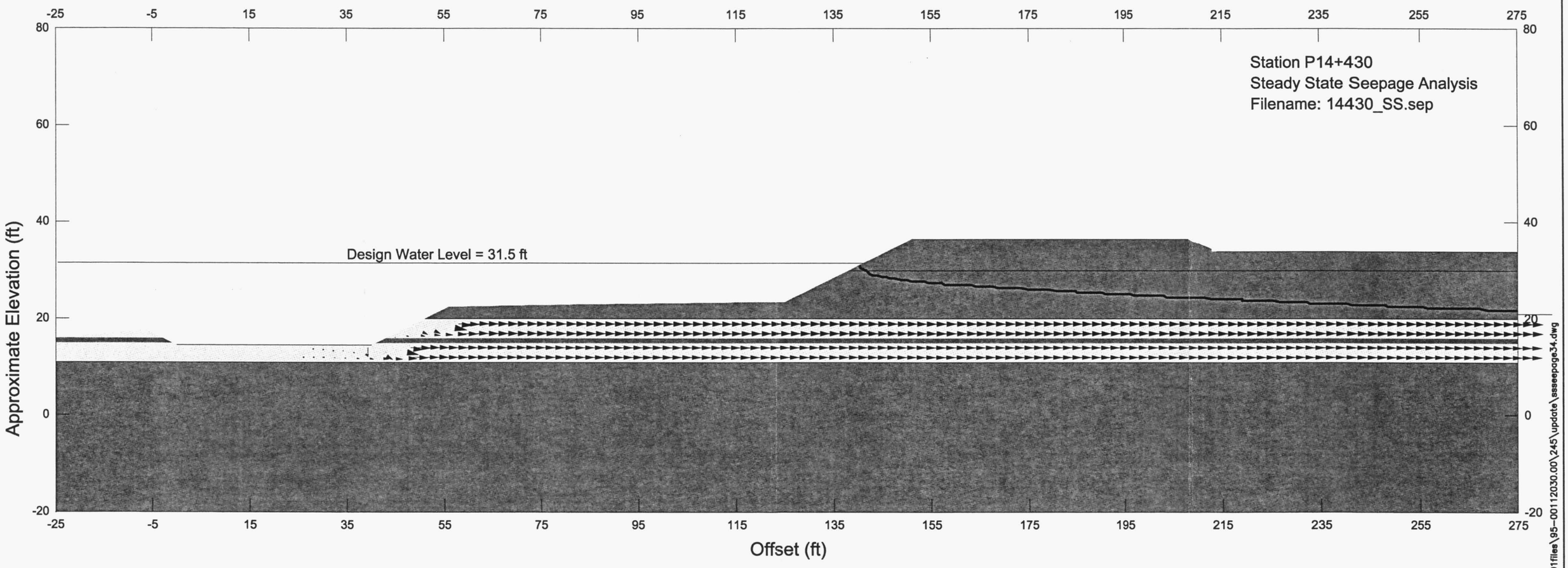
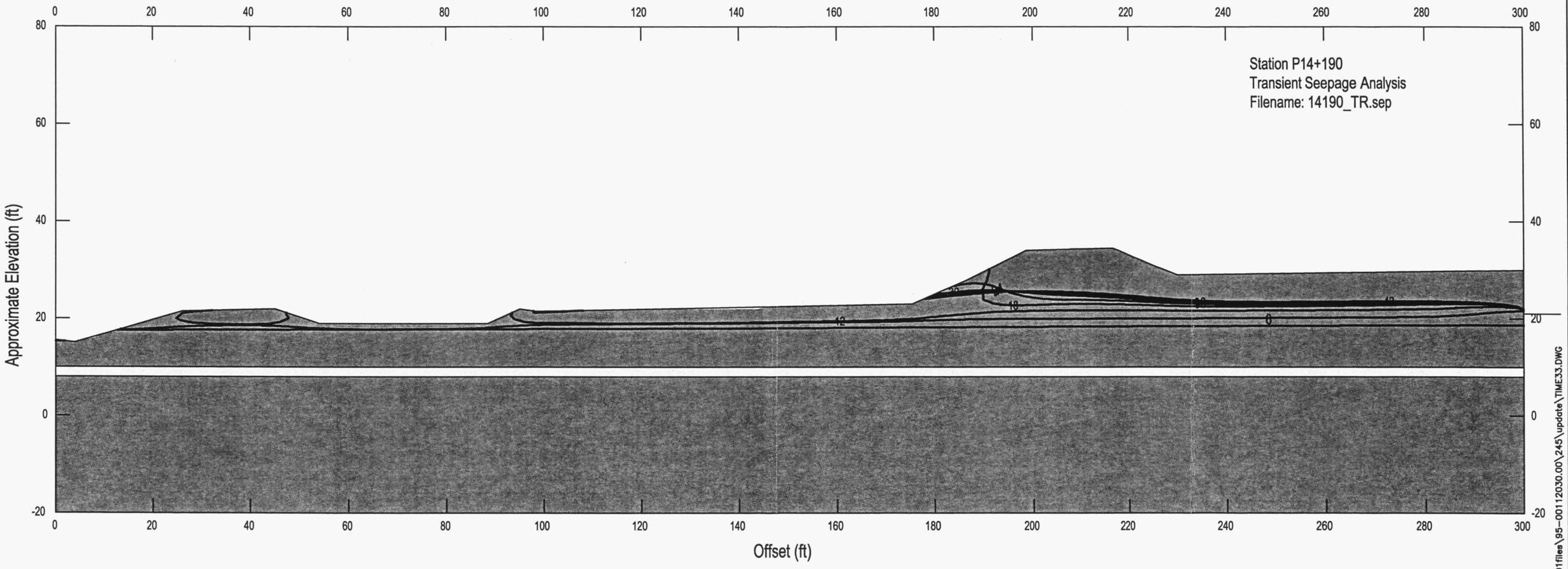
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 33	Figure 98
TEMPOD4 7/15/02	Santa Clara County, California		
	URS		



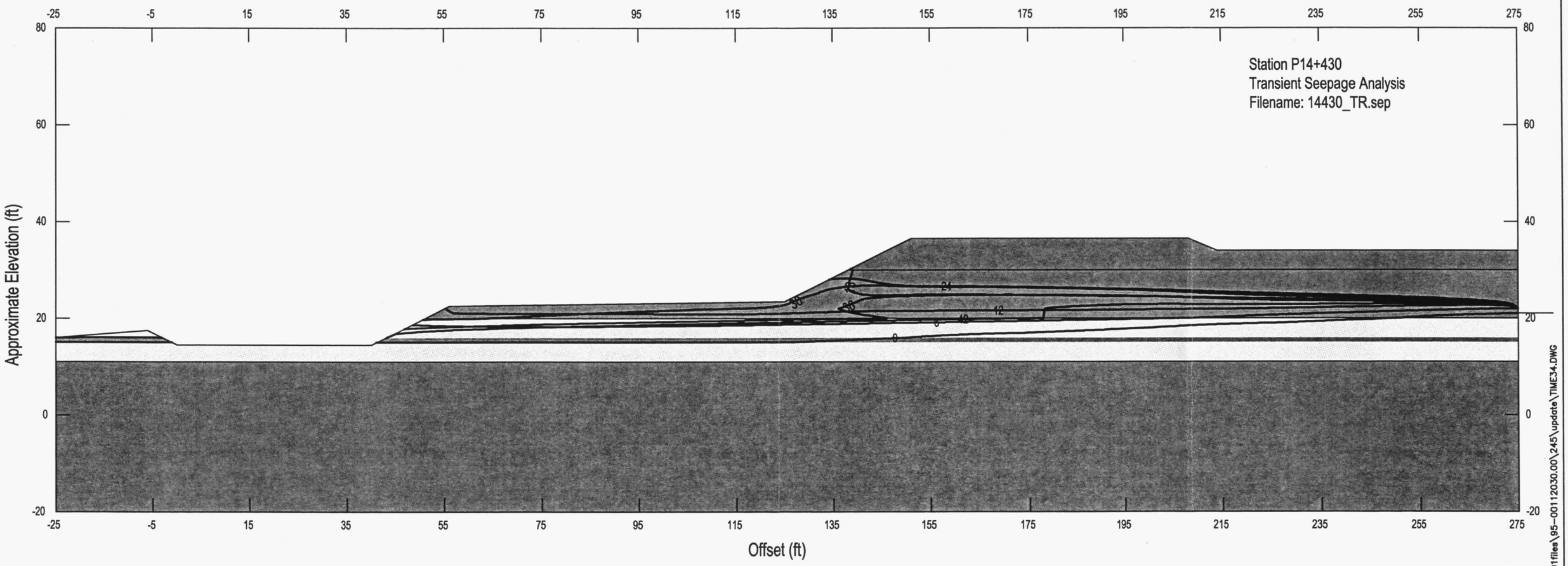
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	STEADY STATE SEEAGE ANALYSIS CROSS SECTION 34	Figure 99
TEMP004 7/15/02	Santa Clara County, California		
	URS		



LEGEND:

36 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 33	Figure 100
7/15/02	URS		

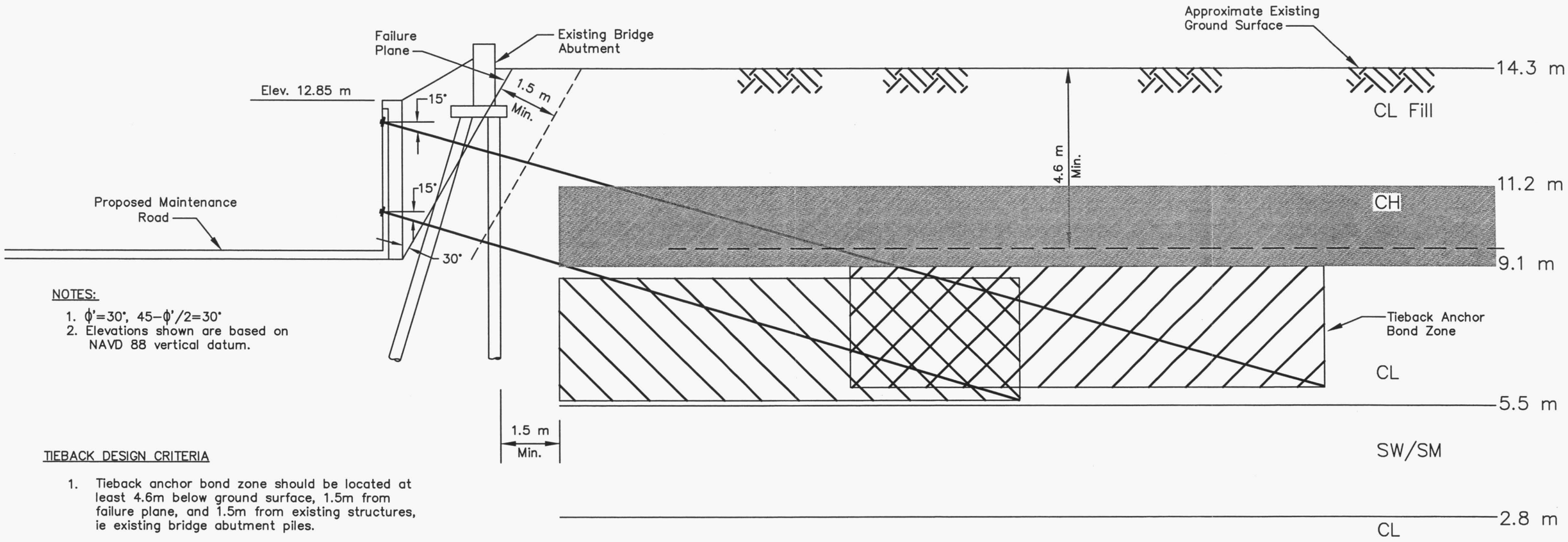


q:\cadd2\01files\95-00112030.00\245\update\TIME34.DWG

LEGEND:

36 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 34	Figure 101
7/15/02	URS		



NOTES:

1. $\phi' = 30°$, $45 - \phi'/2 = 30°$
2. Elevations shown are based on NAVD 88 vertical datum.

TIEBACK DESIGN CRITERIA

1. Tieback anchor bond zone should be located at least 4.6m below ground surface, 1.5m from failure plane, and 1.5m from existing structures, ie existing bridge abutment piles.
2. Tieback length should not exceed 12.2m within tieback anchor bond zone.
3. Tieback anchor bond zone should not extend into Fat Clay layer.
4. Angle or tieback anchors should be between 5 and 15°, while satisfying the above criteria.

TYPICAL SECTION LOOKING SOUTH

Scale 1:100

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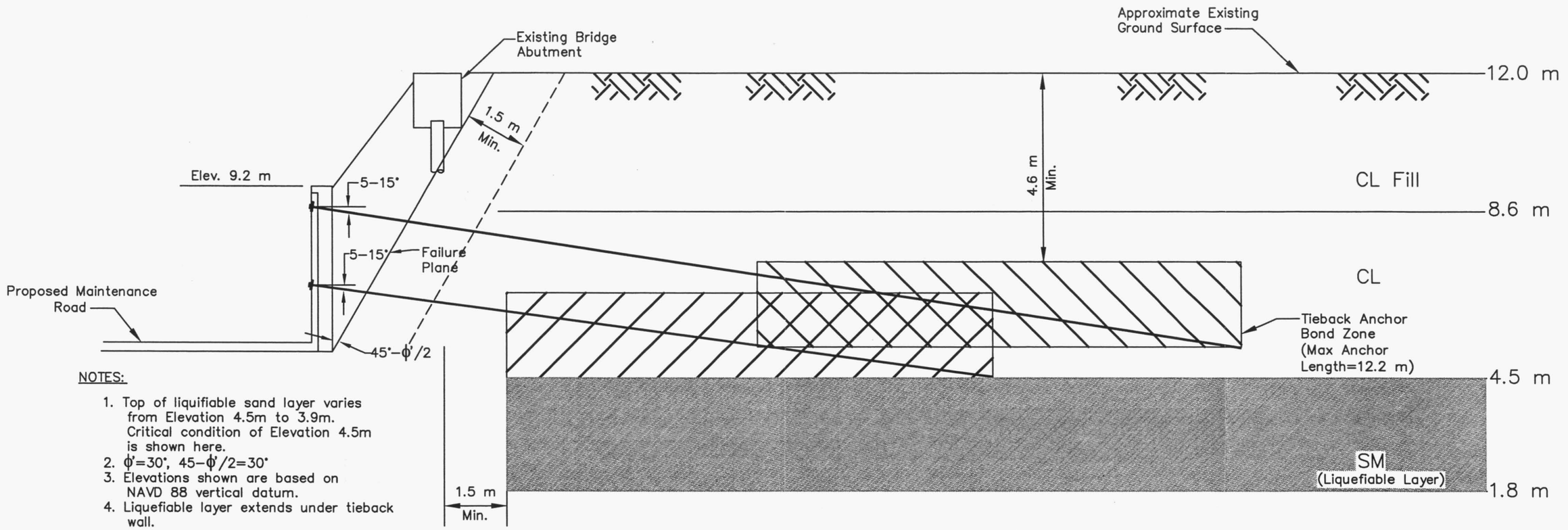
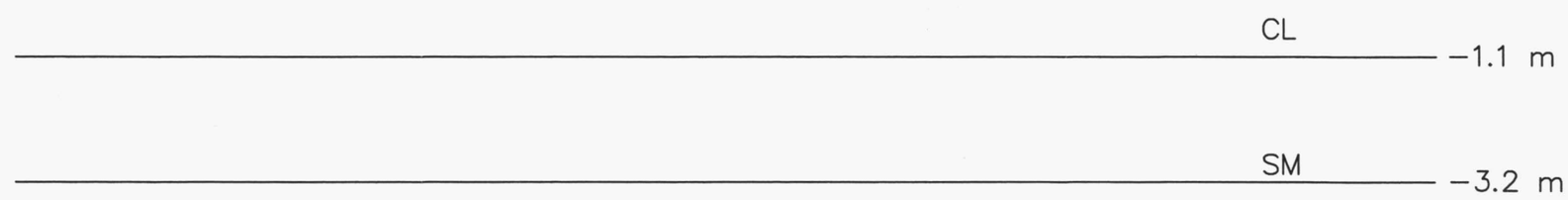
Project No. 95-00112030.00 7/15/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TYPICAL SECTION OF WEST LEVEE AT AIRPORT PARKWAY	Figure 102
URS			



NOTES:

1. Top of liquifiable sand layer varies from Elevation 4.5m to 3.9m. Critical condition of Elevation 4.5m is shown here.
2. $\phi'=30°$, $45-\phi'/2=30°$
3. Elevations shown are based on NAVD 88 vertical datum.
4. Liquefiable layer extends under tieback wall.



TIEBACK DESIGN CRITERIA

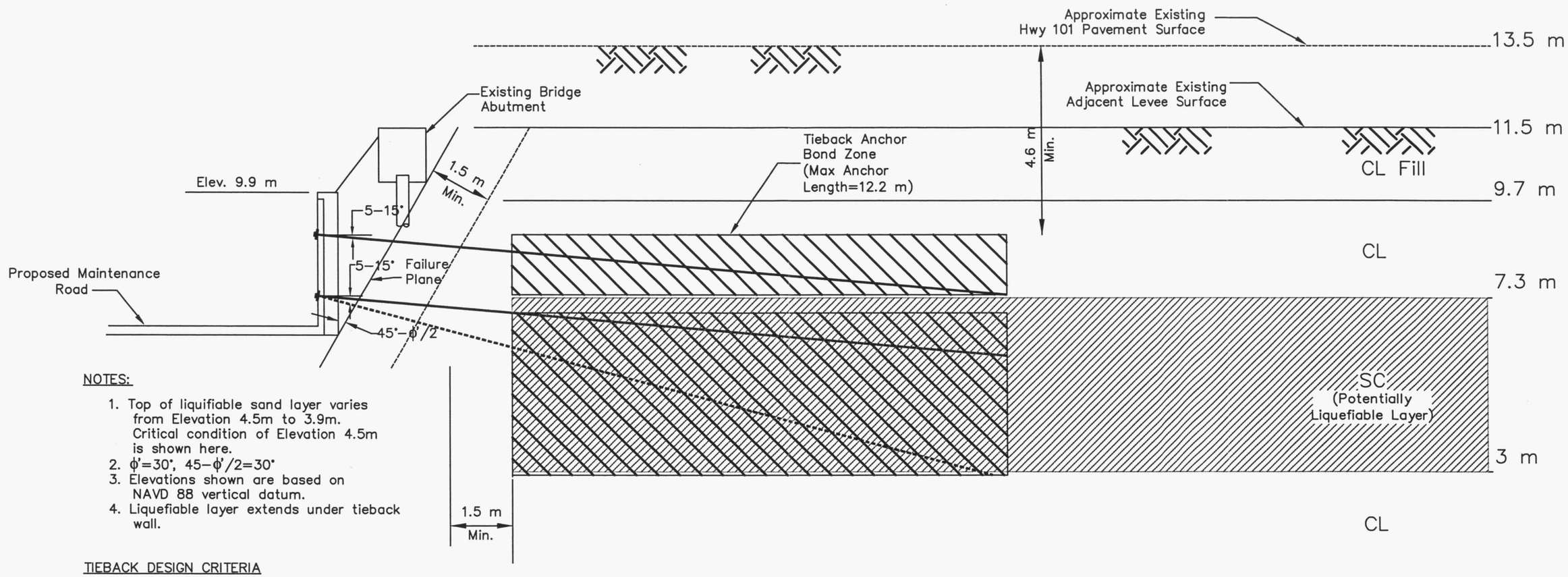
1. Tieback anchor bond zone should be located at least 4.6m below ground surface, 1.5m from failure plane, and 1.5m from existing structures, ie existing bridge abutment piles.
2. Tieback length should not exceed 12.2m within tieback anchor bond zone.
3. Tieback anchor bond zone should not extend into liquefiable sand layer.
4. Angle or tieback anchors should be between 5 and 15°, while satisfying the above criteria.

TYPICAL SECTION LOOKING NORTH

SCALE: 1:100

q:\cadd2\01files\95–00112030.00\245\update\typ–sect101.dwg

Project No. 95–00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	TYPICAL SECTION OF TIEBACK WALL AT EAST LEVEE AT HIGHWAY 101	Figure 103
7/15/02	Santa Clara County, California		
	URS		



NOTES:

1. Top of liquifiable sand layer varies from Elevation 4.5m to 3.9m. Critical condition of Elevation 4.5m is shown here.
2. $\phi'=30°$, $45-\phi'/2=30°$
3. Elevations shown are based on NAVD 88 vertical datum.
4. Liquefiable layer extends under tieback wall.

TIEBACK DESIGN CRITERIA

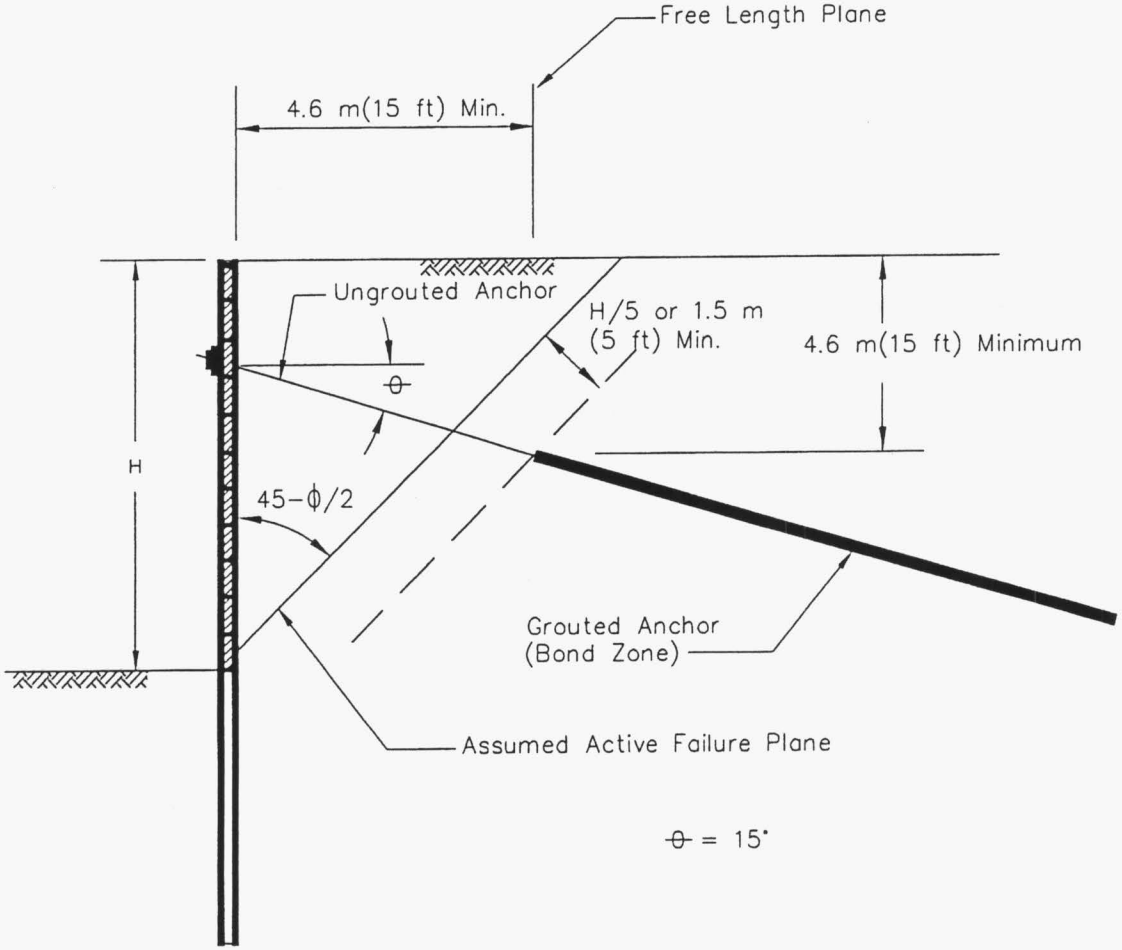
1. Tieback anchor bond zone should be located at least 4.6m below ground surface, 1.5m from failure plane, and 1.5m from existing structures, ie existing bridge abutment piles.
2. Tieback length should not exceed 12.2m within tieback anchor bond zone.
3. Tieback anchor bond zone should not extend into liquefiable sand layer.
4. Angle or tieback anchors should be between 5 and 15°, while satisfying the above criteria.

TYPICAL SECTION LOOKING SOUTH

SCALE: 1:100

q:\cadd2\01files\95-00112030.00\245\typ-sect101West.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT	TYPICAL SECTION OF TIEBACK WALL AT WEST LEVEE AT HIGHWAY 101	Figure 104
7/15/02	Santa Clara County, California		
	URS		



NOTE:
Section shown is for a typical tieback wall.
Refer to Figures 102 through 104 for project specifics.

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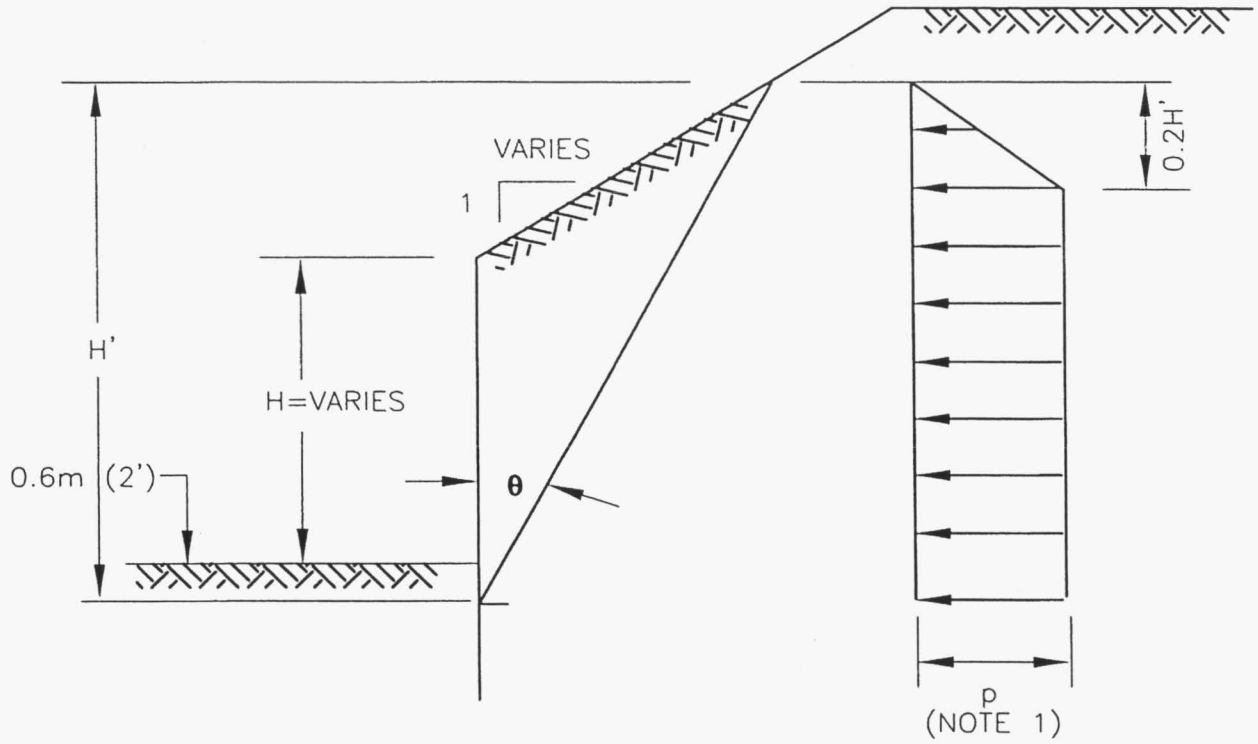
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TYPICAL TIEBACK INSTALLATION	Figure 105
7/15/02			
URS			



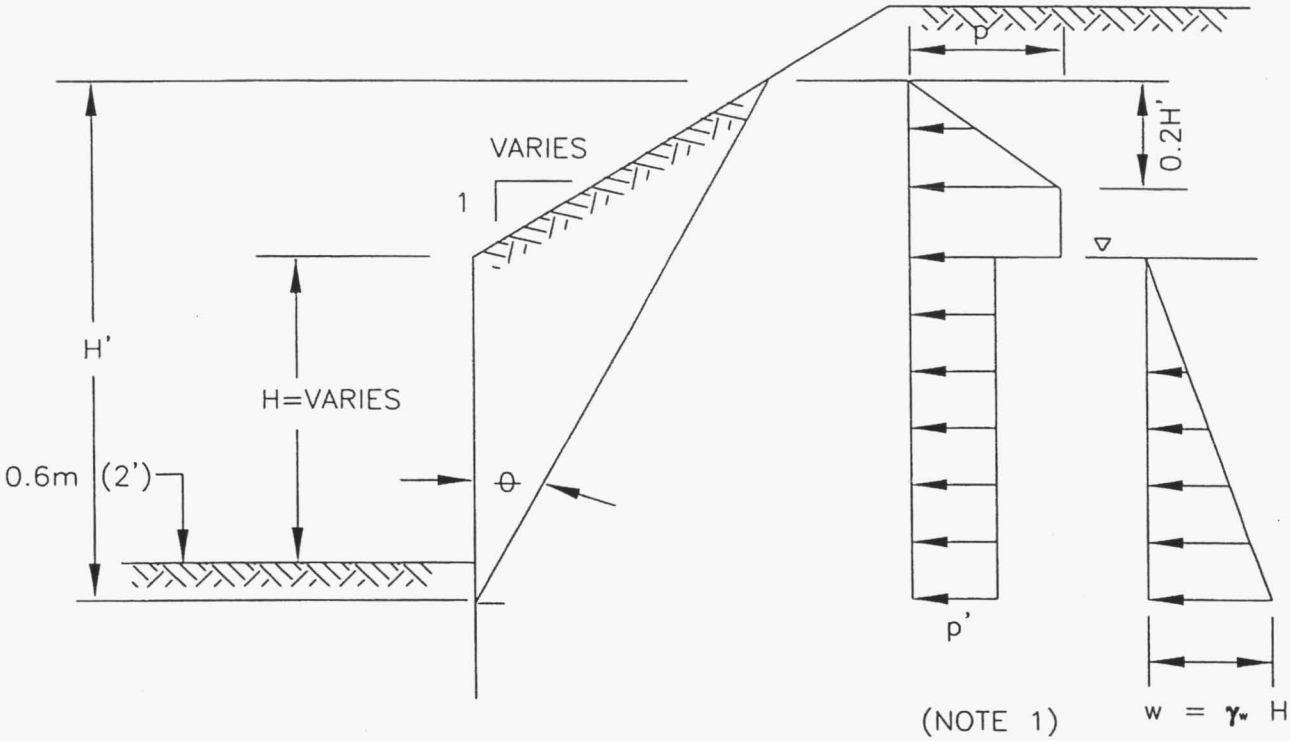
NOTES:

1. STATIC EARTH PRESSURE EQUAL TO p (psf) AS DEFINED IN SECTION 10.6
2. ESTIMATED ϕ VARIES
3. $\theta = 45^\circ - \phi/2$

Q:\CADD2\01FILES\95-00112030.00\245\update\GUADFIG81.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIEBACK DESIGN STATIC EARTH PRESSURE	Figure 106
7/15/02			
URS			

TEMP002



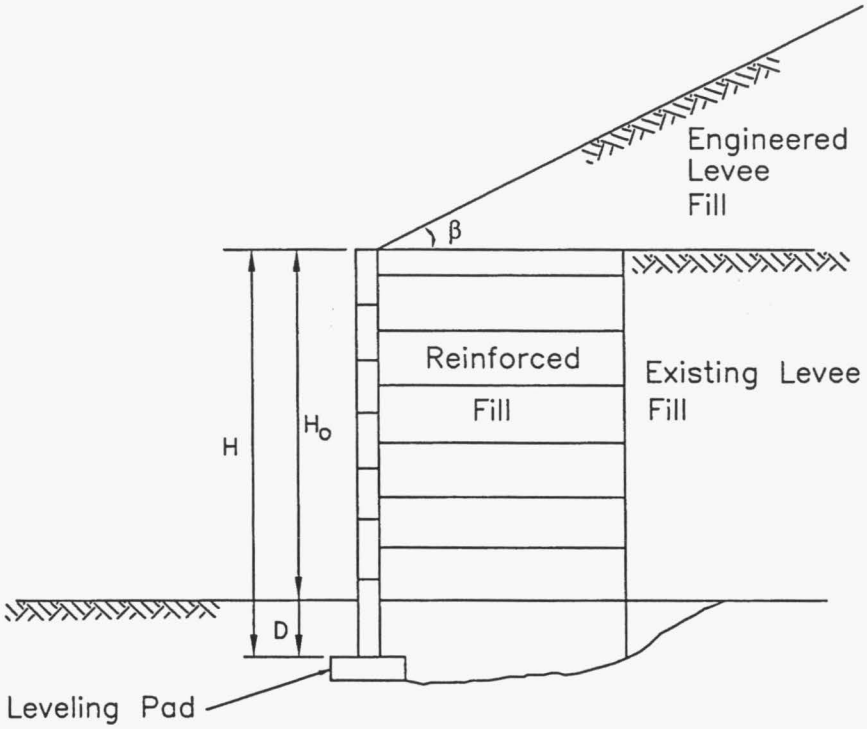
NOTES:

1. STATIC EARTH PRESSURE EQUAL TO p AND p'+w (psf); p' is based on $\gamma-\gamma_w$.
2. ESTIMATED ϕ VARIES
3. $\theta = 45^\circ - \phi/2$
4. ESTIMATED DESIGN HIGH GROUNDWATER

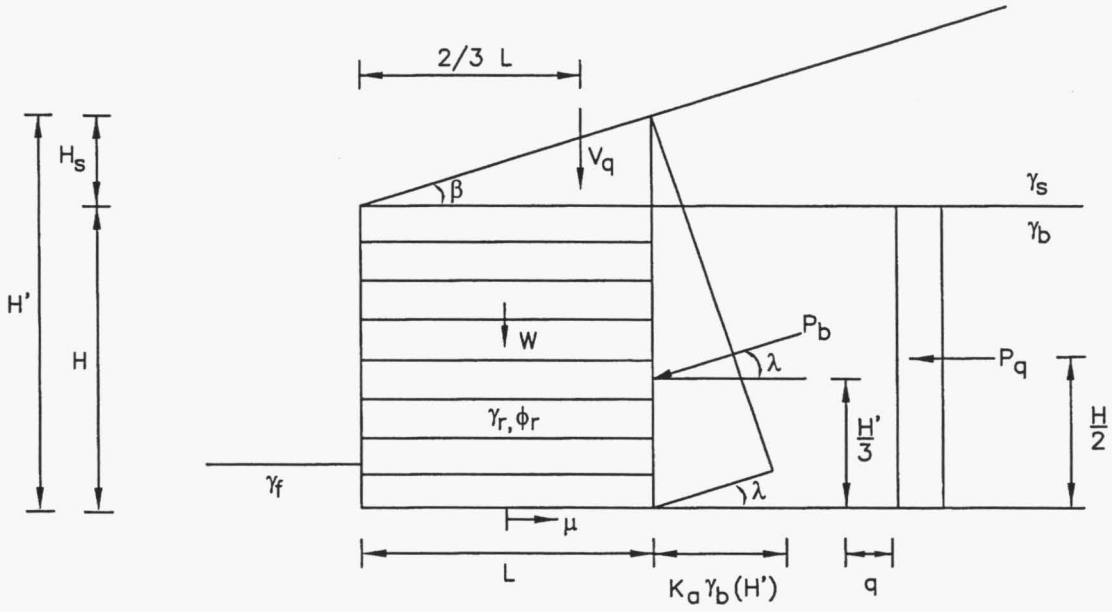
Q:\CADD2\01FILES\95-00112030.00\245\update\GUADFIG107.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIEBACK DESIGN STATIC EARTH PRESSURE	Figure 107
7/15/02			
URS			

TEMP002



MSE WALL CONFIGURATION



$P_b = 0.5 K_a \gamma_b (H')^2$

$P_q = q H$

$\mu = 0.48$

$W = \gamma_r H L$

$V_q = 0.5 \gamma_s H_s L$

$H_s = L \tan \beta$

Assumptions:
(1) $\gamma_b = \gamma_s$
(2) $q = 0.6 \gamma_s$ (kN/m^3)
(3) $\phi_r > 34°$
(4) $\beta = \lambda$ (for extensible reinforcement)

EXTERNAL LOADING DIAGRAM

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Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	TYPICAL MSE WALL CONFIGURATION WITH FORCE DIAGRAM	Figure 108
7/15/02	URS		

TEMP002